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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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MARCH 1937-FEBRUARY 1938

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ABBREVIATIONS

a.—article(s), report(s)

fn.—footnote

r.—reference(s), referred to

r. to p.—reference(s) to publication(s)

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A JOURNAL OF HIGHWAY RESEARCH

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HIGHWAY FILL CONSTRUCTION OPERATIONS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE THEORY OF SOIL CONSOLIDATION AND TESTING OF FOUNDATION SOILS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by L. A. PALMER, Associate Chemist, and E. S. BARBER, Junior Highway Engineer

SEVERAL DEVICES used in performing the compression test described in this report were constructed by the Bureau of Public Roads in 1926 in accordance with plans furnished by Dr. Charles Terzaghi. Investigation of testing technique and practical utilization of the resulting data have been included in the Bureau's program of research on soils for highway purposes continuously since that time.

As part of this program, the Bureau coopcrated with the Massachusetts Institute of Technology in research on soil mechanics from 1926 until the fall of 1932. Dr. Terzaghi was retained by the Bureau as research consultant on the work from the fall of 1926 to the fall of 1929. Dr. Arthur Casagrande was employed by the Bureau as research assistant and was stationed at the Massachusetts Institute of Technology throughout the duration of this cooperative research work.

Other parts of the Bureau's general program were carried on at the Bureau's laboratory at Arlington, Va., and through cooperation with various State highway departments and Federal bureaus.

Special studies have been made at Arlington to determine the magnitude of experimental errors resulting from variables in testing apparatus and procedure, and to determine the agreement between deformations observed in tests with those computed in accordance with the theory of consolidation on which the tests are based.

The compression test and theory of soil consolidation were described by Terzaghi in 1927.[1] Two other reports of distinctive value to those concerned with the practical utilization of the compression test data have also been published.[2]

On the basis of the Bureau's investigations of the test itself, experience in practical use of the test data, and reports just referred to, it is now possible (1) to state definitely that the apparatus and testing procedure described briefly in this report have been found satisfactory, and (2) to present a practical method of estimating that part of the total settlement of soil caused by the loss of water forced vertically out of saturated compressible soil strata in certain soil profiles.

Settlements of soils in profiles differing from those specifically described, or produced by plastic flow, flow of water laterally, or causes not discussed, are beyond the scope of this presentation.

COMPRESSION DEVICE VALUABLE AID IN STUDYING SOIL COMPRESSION

The theory of consolidation of earth materials involves two problems: (1) Determination of the distribution of stresses; and (2) computation of the displacements caused by these stresses. In dealing with many engineering materials and structures the essential problem is to estimate the stresses, since nearly all structural materials possess, within limits, the property of elasticity. With soils, however, the problem is quite complicated for they have both plastic and elastic properties; the materials are essentially heterogeneous; and water is usually present. If the water is free to flow from the compressed soil most of the settlement results directly from the reduction in voids between solid particles, an occurrence that, in this paper, is considered apart from elastic and plastic deformations.

One of the most useful tools in the study of soil consolidation is the laboratory compression device designed by Dr. Terzaghi. The compression tests indicate within a comparatively short time in the laboratory the consolidation characteristics of soils upon which depend the rates of settlement and total settlements of foundations. Thus, compression tests supplant guesswork and "rule of thumb" methods with reliable information for use in the design of foundations.

Figure 1 illustrates the essential features of the testing device used by the Bureau of Public Roads. An undisturbed sample of soil with natural structure and moisture content, as nearly as can be obtained, is carefully placed between two porous stones, which act as filters during the test. Pressures are then applied, ranging from zero to the maximums that are expected in service. A burette connected with the apparatus as shown in figure 1 affords a means of determining the permeability of the sample at any time during the test.

The sample during test is representative of a stratum of saturated, compressible soil in the earth's crust, sandwiched between two layers of sand or other relatively permeable material.

The total amount of consolidation and the speed at which it takes place are controlled by: (1) The moisture content of the soil; (2) the velocity with which water flows vertically through the pores of the sample to the filters above and below in the laboratory, or through the compressible layer to the sand strata in the field; and (3) the frictional resistance of the soil particles to consolidation.

At the instant of load application, the water in the compressible sample or layer is considered as furnishing all of the resistance to consolidation. Thus, if a suitable pressure gauge were connected with the sample in the laboratory, it would show that, at the instant of loading, the water pressure in the sample is practically equal to the pressure applied to the sample. It was stated during the first conference on soil mechanics and foundation engineering, held at Harvard University in June, 1936, that experiments in Sweden disclosed that the water pressures in compressible understrata were approximately equal to the pressures produced by the weight of superimposed fill materials just at the time construction of the fill was completed.

As the pressure on the soil continues, water escapes from the sample or field stratum, causing the pressure to be gradually transferred to the soil particles or

[1] Principles of Final Soil Classification, PUBLIC ROADS, vol. 8, no. 3, May 1927.
[2] Report of the Special Committee on Earths and Foundations, Proceedings of the American Society of Civil Engineers, vol. 59, no. 5, May 1933, and discussion by William P. Kimball, Proceedings of the American Society of Civil Engineers, vol. 59, no. 6, August 1933.

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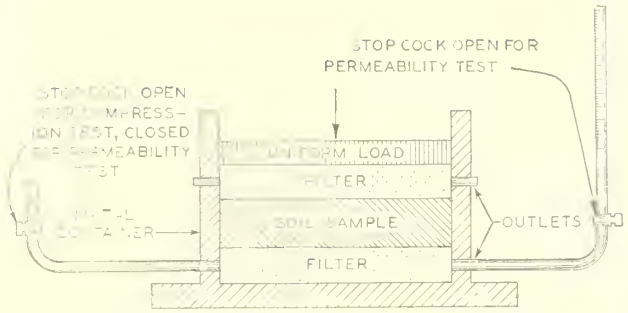

FIGURE 1.—DEVICE FOR DETERMINING THE CONSOLIDATION CHARACTERISTICS OF SOILS IN THE LABORATORY.

FIGURE 2.—PERMEABILITY, COMPRESSION, AND CONSOLIDATION CURVES FOR A SAMPLE OF COMPRESSIBLE SOIL.

"soil skeleton." Eventually, equilibrium is reached and further consolidation ceases when the water pressure becomes equal to zero and the soil skeleton provides all of the resistance to consolidation.

The sample in the compression device is subjected to pressure that is increased by increments. Each increment of pressure acts until apparent equilibrium is attained before the next increment is added. Water escapes under pressure, entering the porous stones and flowing from the outlets as indicated in figure 1.

Consolidation characteristics and permeability of the laboratory specimen are recorded and the data are plotted as shown in figure 2. The load-compression curve shows the voids ratios at which the soil sample attained apparent equilibrium under different pressures. The voids ratio is defined as the ratio (by volume) of voids to solids for a given sample of soil.

Apparent equilibrium in the sample is reached in a relatively short time, and for all practical purposes is true equilibrium. Actually, however, the attainment of true equilibrium would require infinite time. For values of the pressure, p, greater than about one-tenth kilogram per square centimeter, the load-compression curve, as shown, is expressed by the equation,[3]

$$e = 1.050 - 0.442 \log p \quad (1)$$

where e is the voids ratio. The permeability-voids ratio curve (fig. 2, upper) shows the coefficient of permeability of the sample at different stages of compression.

COEFFICIENTS OF PERMEABILITY AND COMPRESSIBILITY DETERMINED

The coefficient of permeability is defined as the velocity of flow when the hydraulic gradient is unity, that is, when the drop in hydrostatic pressure in grams per square centimeter between two points in a soil column equals the distance in centimeters between the points. The usual practice is to record and plot the coefficient of permeability in centimeters per second, as in figure 2. However, for the purpose of simplifying subsequent computations the coefficient of permeability, as used in this report, is expressed in centimeters per minute. The average time-consolidation curve (fig. 2, lower) shows the rate of consolidation under any given pressure. This curve is the average of the individual time-consolidation curves for different load increments. Essential features of the test procedure, the equation for determining the coefficient of permeability, and the use of average time-consolidation curves have been discussed previously.[4]

Both the consolidation characteristics of the soil strata beneath a given footing and the vertical pressures at various depths produced by the superimposed load must be known. In order to solve the many different problems involved in computing earth pressures, one must be familiar with the mathematical theory applicable to all types of problems requiring the calculation of earth stresses and their distribution. The treatment of special cases is then simplified.

The coefficient of permeability, k, is the velocity of flow at a hydraulic gradient of unity. If the thickness of the soil sample, with a voids ratio of e, is d, and if the voids ratio could be reduced to zero, the thickness would become d_0, that is,

$$d = d_0(1+e) \quad (2)$$

The rate at which water would permeate the soil sample if the difference between the two water levels were d_0, rather than d, is designated as k_0. It has been shown [1] that the rate of permeation increases as the hydraulic gradient increases so that from this fact and equation (2),

$$\frac{k}{k_0} = \frac{d}{d_0} = 1+e \quad (3)$$

that is,

$$k_0 = \frac{k}{1+e} \quad (4)$$

[1] Principles of Final Soil Classification, by Dr. Charles Terzaghi, PUBLIC ROADS, vol. 8, no. 3, May 1927.

[3] When plotted to a semilogarithmic scale, e being plotted to the natural scale, the load-compression curve is generally a straight line for loads greater than 0.1 kilogram per square centimeter. The general equation for the straight-line portion of the curve is

$$e = B - Z \log p$$

where B is the intercept of the straight-line portion of the curve at the 1.0-kilogram-per-square-centimeter ordinate and $-Z$ is the slope of the semilog plot of the curve, or range in value of e for unit change of $\log p$.

[4] A Method of Predicting Settlement of Fills Placed on Muck Beds, by F. A. Robeson, PUBLIC ROADS, vol. 16, no. 12, February 1936.

It is convenient to take e as the average voids ratio of a soil sample compressed from an initial voids ratio, e_1, to an equilibrium value e_2, so that

$$e=\frac{e_1+e_2}{2} \quad \text{(5)}$$

On substitution of this value for e in equation (4), one obtains

$$k_0=\frac{2k}{2+e_1+e_2} \quad \text{(6)}$$

The coefficient of compressibility, a, is the average decrease in voids ratio per unit increase of pressure. If the initial voids ratio, e_1, is the condition at equilibrium under a pressure of p_1 grams per square centimeter and e_2 is the corresponding condition at a pressure p_2 grams per square centimeter, then

$$a=\frac{e_1-e_2}{p_2-p_1} \quad \text{(7)}$$

The values of both a and k decrease with the voids ratio, but at different rates. However, the change in the value of the ratio, $\frac{k}{a}$, is fairly small. The ratio, $\frac{k_0}{a}$, is called the coefficient of consolidation, c. From equations (6) and (7) there results from substitution for k_0 and a,

$$c=\frac{k_0}{a}=\frac{p_2-p_1}{e_1-e_2}\times\frac{2k}{2+e_1+e_2} \quad \text{(8)}$$

The utility of this equation may be illustrated by use of the data plotted in the upper curves of figure 2.

From this curve, the pressures, p_1 and p_2, corresponding to $e_1=1.00$ and $e_2=0.80$, are 1,300 and 3,680 grams per square centimeter, respectively. The average voids ratio is 0.90 and from the permeability curve, the coefficient of permeability corresponding to a voids ratio of 0.90 is 4.6×10^{-8} centimeters per second.

Then $$c=\frac{3{,}680-1{,}300}{1.00-0.80}\times\frac{9.2\times10^{-8}}{2+1.00+0.80}\times60=0.0173$$

centimeter per minute.

Similar computations can be made, taking different values for e_1 and e_2 and the corresponding pressures, p_1 and p_2 from the curve, finding the coefficient of permeability corresponding to the average voids ratio as illustrated above, and substituting these values in equation (8).[5] By this procedure, the values of c given in table 1 are computed. These values are plotted against the corresponding voids ratios in figure 3.

[5] The pressures for voids ratios of 0.6, 1.2, and 1.4 were computed from the relation, $e=1.050-0.442 \log p$.

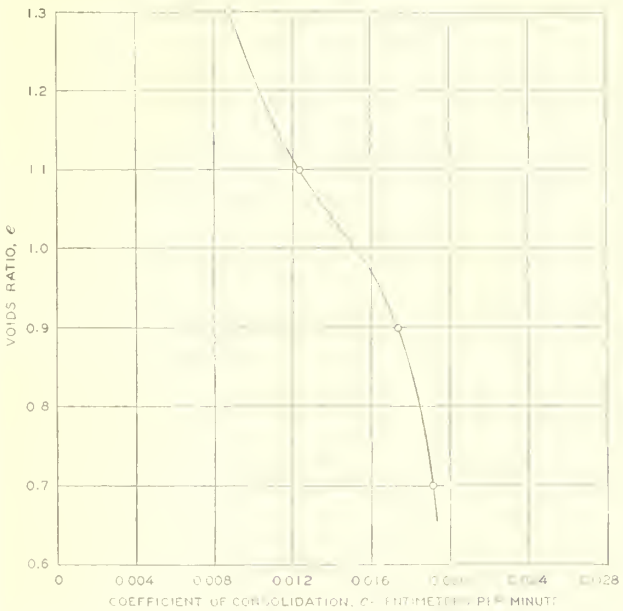

FIGURE 3.—RELATIONS BETWEEN COEFFICIENT OF CONSOLIDATION AND VOIDS RATIO.

TABLE 1.—*Computed values of the coefficient of consolidation, c*

Voids ratios		Pressures corresponding to voids ratios		Coefficient of permeability, k, corresponding to $\frac{e_1+e_2}{2}$	Coefficient of consolidation, c
e_1	e_2	p_1	p_2		
		Grams per square centimeter	*Grams per square centimeter*	*Centimeters per second*	*Centimeters per minute*
1.40	1.20	161	458	22.6×10^{-8}	0.0088
1.20	1.00	458	1,300	10.2×10^{-8}	.0123
1.00	.80	1,300	3,680	4.6×10^{-8}	.0173
.80	.60	3,680	10,400	1.6×10^{-8}	.0190

This paper introduces only the most essential steps in some of the mathematical derivations used both for the computation of earth pressures and for computation of formulas applicable to the consolidation of relatively impervious, saturated strata caused by these pressures. For readers more concerned with direct application of the fundamental principles and formulas than with their development, a discussion of the use of the theory is presented in detail in part 1 of the paper. The mathematical derivations are included as part 2 for those interested also in the theoretical development.

PART I.—APPLICATION OF THEORY OF SOIL CONSOLIDATION

The manner of applying the test data in practice varies, depending upon the pressure distribution conditions beneath the loaded area, the thickness of the compressible layer, and whether it is free to drain both at top and bottom, as in the compression test, or from only one of the bounding surfaces.

When a load is applied on the top of a compressible soil stratum, conditions may be such that the pressure resulting from the load is the same from top to bottom of the stratum, that is, the pressure distribution is uniform. On the other hand, the conditions may be such that the pressure varies, i. e., increases or decreases as the depth increases. In this paper the pressure change per unit of depth is considered to be either zero or constant. If it is zero, the pressure distribution is uniform; if it is constant, the pressure distribution is either triangular or trapezoidal. The cases of pressure distribution are illustrated in figure 4 and are as follows:

Case 1.—Rectangular pressure distribution. The pressure is uniform from top to bottom of the compressible stratum.

Case 2.—Triangular pressure distribution. The pressure varies from zero at the upper bounding surface of the compressible stratum to a finite value greater than zero at the lower bounding surface.

Case 3.—Triangular pressure distribution. The pressure varies from a finite value greater than zero at the top of the compressible layer to a pressure of zero at the bottom.

Case 4.—Trapezoidal pressure distribution. The pressure increases with increasing depth in the compressible stratum. The pressures at both top and bottom surfaces are finite values, each greater than zero.

Case 5.—Trapezoidal pressure distribution. The pressure diminishes with increasing depth in the compressible stratum. The pressures at both top and bottom surfaces are finite values, each greater than zero.

FORMULAS DEVELOPED FOR VARIOUS TYPES OF LOADED AREAS

The pressure distribution throughout the compressible layer depends on the shape and magnitude of the loaded area at the ground surface and on the load distribution over this area. In the examples cited in this paper the load distribution is taken as uniform over the loaded area.

The five types of loaded areas considered are:

A.—A loaded area of great length and width.
B.—A spot load.
C.—A relatively short and narrow, rectangular loaded area.
D.—A relatively narrow loaded area of great length.
E.—A circular loaded area.

The pressure per unit area at any given depth below a loaded area of great extent is taken as equal to the unit pressure on the loaded surface plus the weight of superimposed earth above the point in question. No special formula is required for computations in this simple case.

To compute the pressure below a spot load, use is made of the equation,

$$p_z = \frac{KP}{z^2} \quad \text{---------- (9)}$$

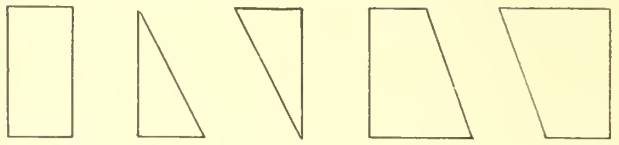

FIGURE 4.—FIVE CASES OF PRESSURE DISTRIBUTION IN SOIL STRATA.

where

$$K = \frac{3}{2\pi\left(1+\frac{r^2}{z^2}\right)^{5/2}} \quad \text{---------- (10)}$$

P = the spot load; and

p_z = the vertical pressure per unit area at a vertical distance, z, below a spot load on the surface and at a horizontal distance, r, from the load.

For any distance on the vertical axis below the load, r is zero and $K = \frac{3}{2\pi}$.

Equations (9) and (10) can be used to determine the vertical pressure produced by any load, uniformly distributed over a rectangular area, provided the ratio of the longer side of the rectangle to z, the vertical depth to the point at which p_z is to be computed, is not greater than one-half. This condition is illustrated in figure 5. If, however, this ratio exceeds one-half, then the procedure is to subdivide the rectangle into rectangular elements (all of equal area) such that the ratio of the longer side of a single element to the vertical distance to the point in question is not greater than one-half. If the point at which it is desired to know the vertical pressure is at a very considerable depth below the loaded footing, then the procedure of subdivision may be unnecessary even in the case of large loaded areas. For example, the base of the Washington Monument is 125 feet square. At any point 250 feet beneath the base the ratio is $\frac{125}{250} = \frac{1}{2}$ and hence the entire loaded area may be considered as a point load at this depth.

When subdivision is necessary it is considered that the load on each element is concentrated at its center. To determine the unit pressure on the vertical center axis, the horizontal distance r from the center of each element to the center of the footing is computed and the ratio, $\frac{r}{z}$, is evaluated. This is substituted in the expression for K, equation (10), and K is computed for the element of area. Having done this with all of the different elements of area, the K values are added together and the sum is substituted in equation (9), the value for P in this equation being the total load on each individual subdivision of area. This procedure will be further illustrated later.

For a relatively narrow loaded area of great length, take a vertical cross section of the loaded strip. At a point on the vertical center axis, p_z is computed from the equation,

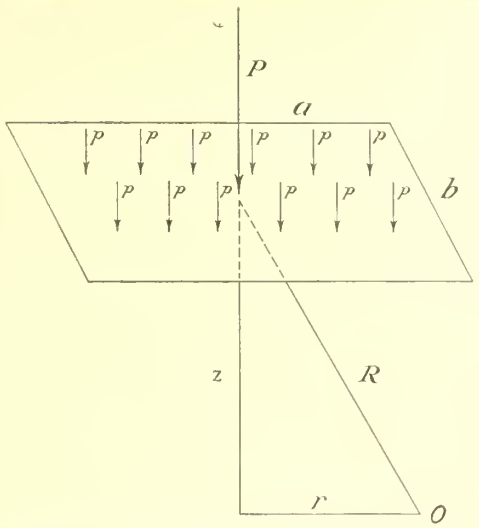

FIGURE 5.—RECTANGULAR AREA LOADED UNIFORMLY. (AREA OF TYPE C, ILLUSTRATING THE PRINCIPLES OF BOTH TYPES B AND C.)

$$p_z=\frac{p}{\pi}(\alpha+\sin\alpha) \qquad (11)$$

where α = twice the angle, in radians, whose tangent is half the width of the strip divided by the vertical distance z to the point in question (see fig. 6); and p = the load per unit area of the footing.

To compute the maximum vertical pressure, p_z, at any point below a circular footing and on the center line (see fig. 6), use the equation,

$$p_z=p\,(1-\cos^3\beta) \qquad (12)$$

where p = the load per unit area of the footing

and β = the angle whose tangent is $\frac{r}{z}$ (see fig. 6).

RATE OF SETTLEMENT COMPUTED BY SIMPLE FORMULAS

If the compressible layer is sandwiched between two permeable sand-gravel layers, one above and one below, water can escape in two directions. If the compressible layer is overlaid with sand and rests on rock, water can escape from but one face of the stratum. There is a double filter in the first case and a single filter in the other. In either case, under a given pressure the thickness of the compressible layer is reduced to the same extent, ultimately. However, the rates of reduction in thickness of the compressible layer will obviously be different in the two cases. Consequently, the settlement during a relatively short time, say in 6 months or a year, will be greater if filtration can occur at both the upper and lower boundaries of the compressible stratum. The filtration effects a diminution in thickness and the earth's surface above the compressible stratum settles.

The total amount of vertical consolidation Q is computed by substitution of the data shown by the load-compression curve in the following expressions:

For soil sample,

$$Q=\left[\frac{e_1-e_2}{1+e_1}\right]d_1 \qquad (13)$$

for soil stratum,

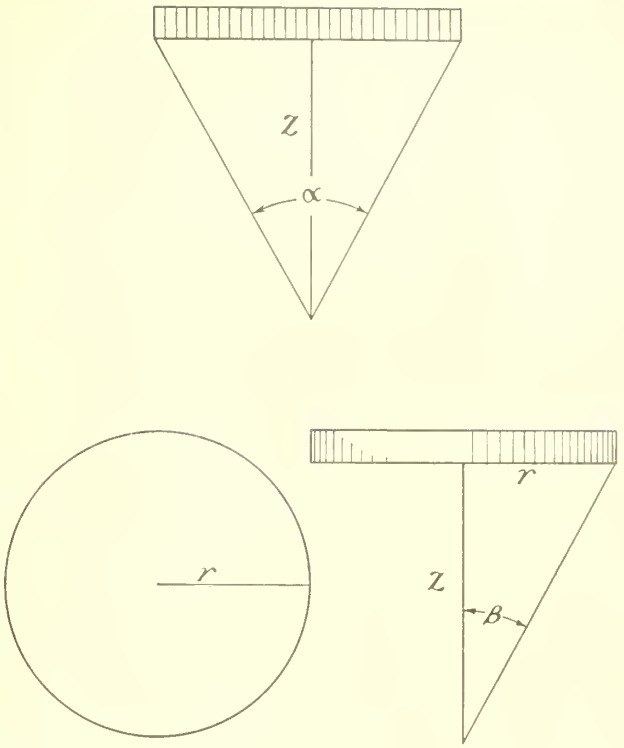

FIGURE 6.—TYPES OF LOADED AREAS. UPPER, VERTICAL CROSS-SECTION OF A STRIP OF GREAT LENGTH (LOADED AREA OF TYPE D). LOWER, A CIRCULAR LOADED AREA (TYPE E).

$$Q=\left[\frac{e_1-e_2}{1+e_1}\right]D_1 \qquad (14)$$

where e_1 = voids ratio of the sample, taken as the average voids ratio in the compressible stratum prior to the application of additional load at the surface;

e_2 = voids ratio of the sample taken as the average voids ratio in the compressible stratum upon reaching equilibrium after the application of additional load at the surface;

d_1 = thickness of soil sample at apparent equilibrium at voids ratio, e_1; and

D_1 = the thickness of the soil stratum at voids ratio, e_1.

The rates at which soil strata settle are computed from test data and field observations by simple formulas.

The theoretical time-settlement relation for soil samples or strata sandwiched between two porous layers and with either rectangular, triangular, or trapezoidal pressure distribution or for strata with rectangular pressure distribution and with a porous layer only at the upper face is given by the expression,

$$q_1=1-\frac{8}{\pi^2}\left(e^{-N}+\frac{1}{9}e^{-9N}+\frac{1}{25}e^{-25N}+\ \ldots\right) \qquad (15)$$

where q_1 = percentage of ultimate consolidation at the end of a definite period of time, t, of load application;

e = Naperian base.

The exponent, N, is related to the thickness of the soil sample, the period of load application, t, and the speed of egress of water from the soil sample or stratum.

Experience has shown that the time-consolidation curves furnished by the compression test on compressible soils (see fig. 2, lower curve) conform in shape with the theoretical curve of equation (15) from q_1 equals zero to q_1 equals 90 percent on the theoretical curve. The experimental values of q_1 attributable to permeability alone, are usually from 60 to 100 percent of the actual ultimate consolidation as obtained in the laboratory with the given loading conditions. Only laboratory values of q_1 within the range in which the experimental curve agrees with the curve showing the theoretical relationship are valid for computations involving the use of equation (15). Beyond the limit of consolidation as effected by the egress of water, further consolidation of most soils is caused by particle rearrangement and does not result from permeability as assumed in the derivation of equation (15).

TIME-CONSOLIDATION RELATIONS GIVEN FOR UNIFORM, TRIANGULAR, AND TRAPEZOIDAL PRESSURE DISTRIBUTIONS

Considering a laboratory sample, the equation for the exponent N is

$$N=\frac{\pi^2 ct}{4\left(\frac{d_0}{2}\right)^2}.$$

When a definite numerical value is assigned to q_1, N has a numerical value, designated as N_1. The expression then becomes

$$t=\frac{N_1\left(\frac{d_0}{2}\right)^2}{2.47c} \quad \text{(16)}$$

where t = period of load application, in minutes.
d_0 = thickness in centimeters that the laboratory sample would have if it were possible for the voids ratio to equal zero.
d_0 = twice the maximum distance water travels when flowing from a sample of d_0 thickness sandwiched between two filters.
c = coefficient of consolidation of compressible soil, in centimeters per minute.

To express t of equation (16) in years instead of minutes, and d_0 in feet instead of centimeters, the following computations are necessary:

$$1 \text{ centimeter}=\frac{1}{2.54\times 12} \text{ feet}$$

and

$$1 \text{ minute}=\frac{1}{365.24\times 24\times 60} \text{ years.}$$

Then

$$t=\frac{N_1\left(\frac{d_0}{2}\right)^2}{2.47\,c}\times\frac{(2.54\times 12)^2}{365.24\times 24\times 60}=\frac{N_1\left(\frac{d_0}{2}\right)^2}{1{,}400\,c} \text{ years.}$$

In this equation d_0 is in feet.

For a compressible stratum in the field, sandwiched between two permeable layers of soil, the time required for a given percentage of consolidation to take place is computed from the equation,

$$t=\frac{N_1\left(\frac{D_0}{2}\right)^2}{1{,}400\,c} \text{ years} \quad \text{(17)}$$

where D_0 = the thickness in feet of the stratum at $e=0$.

When a compressible soil stratum rests on rock and is covered with a porous layer, and when the pressure distribution is uniform, the time-consolidation relation is

$$t=\frac{N_1 D_0^2}{1{,}400\,c} \text{ years} \quad \text{(18)}$$

When a soil stratum rests on impermeable rock and is covered with a porous layer, and the pressure distribution is triangular (case 2), the expression becomes

$$t=\frac{N_2 D_0^2}{1{,}400\,c} \text{ years} \quad \text{(19)}$$

where N_2 is a definite numerical value corresponding to an assigned value for q_2, the percentage consolidation at the time, t, for this type of pressure distribution and boundary conditions. For values of q_1 and q_2, the corresponding numerical values of N_1 and N_2 are given in table 2.

For triangular pressure distribution (case 3) with a permeable layer only at the upper bounding surface of the compressible stratum, the time-settlement relation is

$$t=\frac{N_3 D_0^2}{1{,}400\,c} \text{ years} \quad \text{(20)}$$

For values of q_3 the corresponding numerical values of N_3 are given in table 2. Here q_3 denotes the percentage of consolidation at any time, t, with this type of pressure distribution (case 3) and with the stated boundary conditions.

The procedures followed in computing the theoretical values of N_1, N_2, and N_3, corresponding to values of q_1, q_2, and q_3 will be shown on pages 7 and 8 preceding the illustrative examples.

TABLE 2.—*Values of N_1, N_2, and N_3 corresponding to values of q_1, q_2, and q_3 for cases 1, 2, and 3, respectively*

Consolidation q_1, q_2, or q_3	Numerical values of—		
	N_1	N_2	N_3
Percent			
0	0.00	0.00	0.00
5	.005	.06	.002
10	.02	.12	.005
15	.04	.18	.01
20	.08	.25	.02
25	.12	.31	.03
30	.17	.39	.06
35	.24	.47	.09
40	.31	.55	.12
45	.39	.63	.17
50	.49	.73	.24
55	.59	.84	.32
60	.71	.95	.42
65	.84	1.08	.54
70	1.00	1.24	.69
75	1.18	1.42	.88
80	1.40	1.64	1.08
85	1.69	1.93	1.36
90	2.09	2.35	1.77
95	2.80	3.02	2.54
100	Infinity	Infinity	Infinity

With trapezoidal pressure distribution, the pressure resulting from the applied load may either increase (case 4) or decrease (case 5) with depth, and has a finite value at both the upper and lower boundaries or surfaces of the compressible stratum. The ratio of the pressure at the upper boundary to that at the lower is designated as u. The value of u is unity for uniform and either zero or infinity for triangular pressure dis-

tribution. For trapezoidal pressure distribution, (case 4), u is less than 1 and greater than zero. An interpolation factor, J, is applied in this case in computing the rate of settlement if the compressible stratum is overlaid with sand and rests on rock. When this condition obtains, interpolation is between the two extremes of uniform and triangular pressure distributions. With the compressible stratum bounded below by sand and above by rock, and with a trapezoidal pressure distribution such that u exceeds unity, (case 5) an interpolation factor, J', is used in computing the rate of consolidation.

The expressions for J and J' in terms of u are as follows:

For case 4, $$J=\frac{\log\left[\frac{2(\pi-2)u+4}{\pi(1+u)}\right]}{\log\frac{4}{\pi}} \quad \text{(21)}$$

For case 5, $$J'=\frac{\log\left[\frac{u(\pi-2)+2}{(\pi-2)(u+1)}\right]}{\log\left[\frac{\pi}{2(\pi-2)}\right]} \quad \text{(22)}$$

Values for J and J' corresponding to values assigned to u are listed in table 3.

The time-settlement relations for cases 4 and 5, trapezoidal pressure distributions, are:

Case 4, $$t=\frac{N_4 D_0^2}{1{,}400\,c} \text{ years} \quad \text{(23)}$$

Case 5, $$t=\frac{N_5 D_0^2}{1{,}400\,c} \text{ years} \quad \text{(24)}$$

where $$N_4=N_1+J(N_2-N_1) \quad \text{(25)}$$

and $$N_5=N_3+J'(N_1-N_3) \quad \text{(26)}$$

TABLE 3.—*Values of u, J and J' used in computing the rates of settlement for the two types of trapezoidal pressure distribution*

Pressure increasing with depth, u varying from 0 to 1		Pressure diminishing with depth, u varying from 1 to infinity	
Value of u	Value of J	Value of u	Value of J'
0.0	1.00	1.0	1.00
		1.2	.92
.1	.84	1.4	.86
		1.6	.80
.2	.69	1.8	.75
		1.9	.73
.3	.57	2.0	.71
		2.2	.67
.4	.46	2.5	.62
		3.0	.55
.5	.36	3.5	.50
		4.0	.45
.6	.27	4.5	.42
		5.0	.39
.7	.19	6.0	.34
		7.0	.30
.8	.12	8.0	.27
		9.0	.25
.9	.06	10.0	.23
		12.0	.20
1.0	.00	15.0	.17
		20.0	.13
		100.0	.02

FAIRLY CLOSE AGREEMENT FOUND BETWEEN THEORETICAL AND TEST VALUES

The time-consolidation curve (fig. 2) represents the average curve for pressures of $p=1.079$, 2.118, 4.196, and 8.352 kilograms per square centimeter. The time-consolidation curve for the load $p=0.61$ kilogram per square centimeter was obviously influenced by experimental error and therefore was not included in computing the values for the average curve.

The divergence of the average curve from the curve given by equation (15) may be determined by means of equation (16) as follows:

For different values of q_1, say 25, 50, and 75 percent, the values of t are obtained from the curves. The average values of e at the different stages of loading are also obtained from these curves. Then the values of c, corresponding to the average values of e, are read from the curve in figure 3; the value of d_0 is computed; the numerical values are substituted in equation (16); and the values for N_1 may thus be computed, assuming that the relationship in equation (16) obtains.

The thickness of the sample, d, at a voids ratio, $e=1.670$, was found in the test to be 1.130 centimeters. Therefore the thickness d_0 when $e=0$ would be 0.423 centimeter.

The values for N_1 thus obtained (see table 4) for equal values of q_1 are averaged and found to be 0.11 for $q_1=25$, 0.44 for $q_1=50$, and 1.30 for $q_1=75$.

The theoretical values of N_1 for the same values of q_1 are shown in the last column of table 4. The agreement between the determined and theoretical values is fairly good. The procedure in computing these theoretical values of N_1 follows:

TABLE 4.—*Values for q_1, c, c, and N_1 obtained from laboratory data*

Pressure interval	Consolidation, q_1	Average voids ratio, e	Values of c corresponding to average e	Values of N_1 computed from data [1]	Values of N_1 computed from theory
Kg per sq. cm	*Percent*				
0.610–1.079	25	1.17	0.0168	0.10	0.12
	50	1.13	.0116	.49	.49
	75	1.08	.0140	1.47	1.18
1.079–2.118	25	1.60	.0152	.13	.12
	50	.97	.0159	.42	.49
	75	.94	.0166	1.20	1.18
2.118–4.196	25	.87	.0177	.12	.12
	50	.84	.0180	.42	.49
	75	.80	.0183	1.32	1.18
4.196–8.352	25	.74	.0188	.09	.12
	50	.72	.0189	.44	.49
	75	.69	.0191	1.22	1.18

[1] For $q_1=25$ percent, average N_1 (computed from data in column 5) = 0.11. For $q_1=50$ percent, average N_1 (computed from data in column 5) = 0.44. For $q_1=75$ per cent, average N_1 (computed from data in column 5) = 1.30.

COMPUTATIONS GIVEN FOR VARIOUS TYPES OF PRESSURE DISTRIBUTION AND BOUNDARY CONDITIONS

Case 1.—The computations for case 1 cover uniform pressure distribution, irrespective of boundary conditions, and triangular and trapezoidal pressure distribution when the compressible stratum is bounded on both top and bottom by permeable strata. To compute the value of N_1 corresponding to any definite value assigned to q_1 in the expression,

$$q_1=1-\frac{8}{\pi^2}\left(e^{-N}+\frac{1}{9}e^{-9N}+\frac{1}{25}e^{-25N}+\ldots\right) \quad \text{(15)}$$

one may, as an approximation, neglect all terms of e except the first. Then

$$q_1=1-\frac{8}{\pi^2}e^{-N} \quad \text{(27)}$$

or

$$1-q_1=\frac{8}{\pi^2}e^{-N} \quad \text{(28)}$$

For $q_1=\frac{1}{2}$, equation (28) becomes

$$\frac{1}{2}=\frac{8}{\pi^2}e^{-N_1}$$

and

$$\log_e\frac{1}{2}=\log_e\frac{8}{\pi^2}-N_1,$$

where "$\log_e$" designates the natural logarithm.

$$N_1=\log_e\frac{8}{\pi^2}-\log_e\frac{1}{2}=2.3026\left(\log\frac{8}{\pi^2}-\log\frac{1}{2}\right),$$

where "log" denotes the base 10 system of logarithms. Hence $N_1=0.48$.

The more accurate procedure would be to assign values to N, include several terms of the series, and solve for q_1. In this manner it is found that for $N_1=0.49$ the value of q_1 is 0.50 or 50 percent. (See table 2.) However, it is seen that by neglecting all terms of e except the first in equation (15), the error, $0.49-0.48=0.01$, is small.

Case 2.—Triangular pressure distribution with zero pressure at the upper boundary, when the compressible stratum is bounded by an impermeable stratum below and a permeable stratum above. For this condition the theoretical time-settlement relationship is given by the expression,

$$q_2=1-\frac{32}{\pi^3}\left(e^{-N}-\frac{1}{27}e^{-9N}+\frac{1}{125}e^{-25N}-,+\ldots\right)\quad(29)$$

By neglecting all terms of e except the first, one may write,

$$1-q_2=\frac{32}{\pi^3}e^{-N}\quad(30)$$

for $q_2=\frac{1}{2}$, $\qquad N_2=\log_e\frac{32}{\pi^3}-\log_e\frac{1}{2}=0.72.$

This compares favorably with the value, $N_2=0.73$ for $q_2=50$ percent, table 2, obtained by taking more terms in the series than the first in equation (29).

Case 3.—Triangular pressure distribution with zero pressure at the lower boundary, when the compressible stratum is bounded by an impermeable stratum below and a permeable stratum above. It will be shown in part 2 of this paper that

$$q_3=2q_1-q_2\quad(31)$$

The simplest procedure in computing values of N_3 that correspond to values of q_3 is to take the case, $N_1=N_2=N_3$. For $N_1=N_2=N_3=0.12$, it is seen in table 2 that $q_1=0.25$ and $q_2=0.10$, so that $q_3=2\times0.25-0.10=0.40$, That is for $q_3=40$ percent, $N_3=0.12$.

Case 4.—Trapezoidal pressure distribution with pressure increasing with depth, when the compressible stratum is bounded by an impermeable stratum below and a permeable stratum above. The value, N_4, is computed from equation (25), which is $N_4=N_1+J(N_2-N_1)$, applicable for $q_4=q_1=q_2$. J in turn is computed from equation (21) for any given value of u. For example, when $u=0.50$, $J=0.36$ (see table 3). For $q_1=q_2=q_4=50$ percent, $N_1=0.49$ and $N_2=0.73$. Then

$$N_4=0.49+0.36(0.73-0.49)=0.58.$$

When this value is substituted for N_4 in equation (23),

$$t=\frac{N_4D_0^2}{1{,}400\,c}\text{ years},$$

the time required to produce 50 percent of the ultimate consolidation is easily calculated. The value of c is obtained from laboratory data according to the procedure already indicated and $D_0=\frac{D_1}{1+e_1}$ as already shown (equation 2). The thickness, D_1, of the compressible stratum prior to applying a load at the surface is known from the test borings. If $D_1=20$ feet and $e_1=1.5$, then $D_0=\frac{20}{2.5}=8$ feet. And if $c=0.01$ and $N_4=0.58$, then $t=\frac{0.58\times64}{14}=2.65$ years. This is the time required for 50 percent of the ultimate consolidation to take place under the given conditions, since a value of $N_2=0.58$ corresponds to 50 percent consolidation.

Case 5.—Trapezoidal pressure distribution with pressure decreasing with depth, when the compressible stratum is bounded by an impermeable stratum below and a permeable stratum above. For $q_1=q_3=q_5$, the value, N_5 is computed from equation (26), which is,

$$N_5=N_3+J'(N_1-N_3),$$

for any given value of u. From table 3, $J'=0.71$ when $u=2$. For $q_1=q_3=q_5=50$ percent, $N_1=0.49$ and $N_3=0.24$ from table 2. Then by substitution, $N_5=0.24+0.71(0.49-0.24)=0.42$. Now take the conditions already assumed for case 4, that is, $c=0.01$, $e_1=1.5$, the average voids ratio in the stratum prior to loading, and $D=20$ feet. Then

$$t=\frac{N_5D_0^2}{1{,}400\,c}\text{ years}=\frac{0.42\times64}{14}\text{ or }1.9\text{ years}.$$

DISCUSSION OF CONDITIONS GOVERNING ILLUSTRATIVE EXAMPLES THAT FOLLOW

In the following examples it is assumed that the material composing the compressible stratum has the consolidation characteristics derived from computations of the data plotted in figures 2 and 3, and that the weight per cubic foot of this material as well as that of the sand above it is 100 pounds. Cases 1 to 5 inclusive are linear types of pressure distribution, that is, the pressure distribution diagrams are composed of straight lines. All of these five types of pressure distribution give the same time-consolidation relation, equation (17), when the compressible stratum is bounded both above and below by drainage courses (boundary condition A). Equation (18) is the time-consolidation relation when there is but one drainage course (boundary condition B) and the pressure distribution is rectangular (case 1). When there is but one drainage course, the stratum resting on rock and being overlaid with sand, equations (19), (20), (23), and (24) are the time-consolidation relations for cases 2, 3, 4, and 5, respectively, and computations are made with the aid of tables 2 and 3 and the interpolation formulas, making use of J and J', that is, equations (21), (22), (25), and (26).

Cases 1 to 5 inclusive may be defined in terms of u, the ratio of the pressure at the upper surface of the compressible stratum to the pressure at the lower surface, that is:

Case 1, rectangular pressure distribution, $u=1$.

Case 2, triangular pressure distribution, $u=0$.

Case 3, triangular pressure distribution, $u=$infinity.

Case 4, trapezoidal pressure distribution, u varies from 0 to 1.

Case 5, trapezoidal pressure distribution, u varies from 1 to infinity.

In solving any problem involving case 4, the formulas and principles of cases 1 and 2 are applied. When the pressure distribution is of the type case 5, application is made of the formulas and principles that pertain to cases 1 and 3. Cases 1 and 2 are both limiting conditions of case 4, and cases 1 and 3 are both limiting conditions of case 5. The types of pressure distribution most frequently encountered in foundation problems are cases 1 and 5, more especially 5.

Example 1.—Case 1 pressure distribution; type A loading. Consider a uniform load of 3 tons per square foot over an area of great extent. Immediately beneath the loaded area there is a layer of sand, 10 feet thick. Below the sand is a compressible stratum also 10 feet thick and below this stratum there may be either (A) sand or (B) rock. It is desired to know the total settlement and the time required for 25, 50, and 75 percent of the total settlement to take place.

Use equation (17) for boundary condition A and equation (18) for boundary condition B for the time-consolidation part of the problem. It is assumed that consolidation of the compressible stratum resulting from its own weight and that of the sand above it has reached completion prior to application of the surface load.

Prior to loading, the pressure is 1,000 pounds per square foot at the top and 2,000 pounds per square foot at the bottom of the compressible layer. In figure 2, these pressures correspond to voids ratios of 1.28 and 1.06 respectively. A pressure of 1 kilogram per square centimeter is very nearly 1 ton per square foot so that the units of pressure of figure 2 (load-compression curve) are directly applicable. The average voids ratio prior to loading is then

$$\frac{1.28+1.06}{2}=1.17=e_1.$$

After loading, the pressure at the top of the stratum is 3.5 tons and at the bottom, 4 tons per square foot. These pressures correspond to the voids ratios (see fig. 2), 0.80 and 0.78, respectively, the average, e_2 being 0.79. The total settlement is computed by equation (14), that is,

$$Q=\frac{e_1-e_2}{1+e_1}\times D_1=\frac{1.17-0.79}{1+1.17}\times 10=1.75 \text{ feet.}$$

If there were no voids whatsoever in the material composing the compressible stratum, its thickness would be

$$D_0=\frac{1}{1+e_1}D_1=\frac{1}{2.17}\times 10=4.61 \text{ feet.}$$

The average voids ratio is reduced by the surface loading from $e_1=1.17$ to $e_2=0.79$. Within this interval there is an average e of $\frac{1.17+0.79}{2}=0.98$. From figure 3, a voids ratio of 0.98 corresponds to a value for c, the coefficient of consolidation, equal to 0.0157.

Now apply equation (17) for boundary condition A, the compressible stratum sandwiched between two drainage courses. The times required for 25, 50, and 75 percent of ultimate consolidation (1.75 feet) to take place are, using table 1 for values of N_1,

for $q_1=25$ percent, and $N_1=0.12$, $t=\frac{N_1\left(\frac{D_0}{2}\right)^2}{1{,}400\,c}$

$=\frac{0.12\times\left(\frac{4.61}{2}\right)^2}{1{,}400\times 0.0157}=0.03$ year or 11 days,

for $q_1=50$ percent, $N_1=0.49$, $t=\frac{0.49\times\left(\frac{4.61}{2}\right)^2}{1{,}400\times 0.0157}$

$=0.12$ year $=44$ days,

for $q_1=75$ percent, $N_1=1.18$, $t=\frac{1.18\times\left(\frac{4.61}{2}\right)^2}{1{,}400\times 0.0157}$

$=0.285$ year $=104$ days.

For condition B, the stratum covered with sand and resting on rock, the values of N_1 corresponding to values of q_1 are the same as for condition A. Solving equation (18) for $q_1=25$ percent, $t=\frac{0.12\times(4.61)^2}{1{,}400\times 0.0157}=0.12$ year $=44$ days. Similarly, for $q_1=50$ percent, $t=0.48$ year, and for $q_1=75$ percent, $t=1.14$ years.

Twenty-five percent of 1.75 feet, the ultimate settlement, is 5.2 inches. For condition A, this amount of settlement occurs in 11 days; for condition B, 44 days are required for this amount of settlement.

Example 2.—Case 5 pressure distribution; type C loading. Instead of a loading area of great extent as in example 1, there is a rectangular rigid footing, 10 feet wide and 30 feet long, carrying a uniform load of 3 tons per square foot. This is at the ground surface above the same compressible stratum described in example 1. The average voids ratio, e_1, of the compressible layer prior to loading is taken as 1.17; its thickness prior to loading is 10 feet; and the weight of the sand at the upper level of the stratum is 1,000 pounds per square foot.

The vertical distance, z, of the footing above the upper boundary of the stratum is 10 feet. The ratio of the length of the footing to z is 3. This ratio is too large to warrant taking the uniform load on the footing as a point load at its center. For purposes of computations it is necessary to consider the footing as subdivided into rectangles such that the ratio of the length of a single rectangle to z does not exceed one-half (see figs. 5 and 7). The footing may be divided into twelve 5-foot squares to meet this requirement.

The total load on the footing is $3\times 10\times 30=900$ tons. The load on a single square is 75 tons and it is considered that this load is concentrated at the center of the square. The horizontal distance from the center of a square to the center of the 10- by 30-foot foundation is r and this is found from the properties of a right triangle. For example, the center of square number 5 from C, the center of the footing, is the square root of $\left(\frac{5}{2}\right)^2+\left(\frac{5}{2}\right)^2$, or $\frac{5}{\sqrt{2}}=r$. The same value for r is obtained

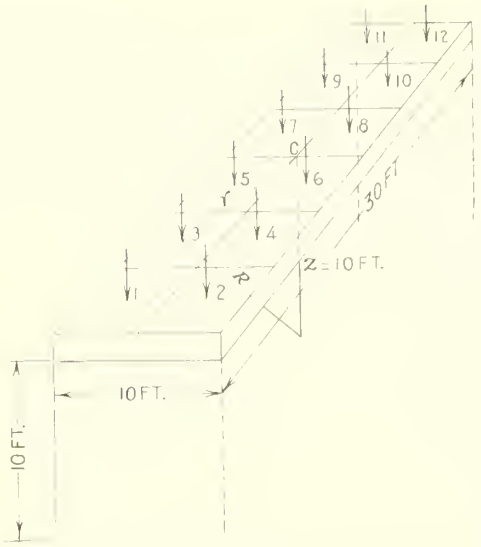

FIGURE 7.—RECTANGULAR LOADED AREA (TYPE C).

for squares 6, 7, and 8. The ratio, $\frac{r}{z}$, appearing in equation (10) is $\frac{5}{10\sqrt{2}}$ or $\frac{1}{2\sqrt{2}}$ for squares 5, 6, 7, and 8. These values and those obtained for the other squares (fig. 7) are given in table 5, together with the value of K obtained by substituting the value for $\frac{r}{z}$ in equation (10),

$$K=\frac{3}{2\pi\left(1+\frac{r^2}{z^2}\right)^{5/2}}.$$

The three values for K, corresponding to the three groups of load numbers given in the first column of table 5, are added together, their sum being 0.5404. This must be multiplied by 4 to give K for the entire footing. The vertical stress at a point 10 feet below C and on the vertical center axis through C is then calculated from equation (9). That is,

$$p_z=\frac{KP}{z^2}=\frac{75}{100}\times 4\times 0.5404=1.62 \text{ tons per square foot.}$$

TABLE 5. *Computations of vertical stresses below a rectangular footing 10 by 30 feet*

Load numbers (see fig. 7)	Distance, r	Ratio, $\frac{r}{z}$	Coefficient K
	Feet		
5, 6, 7, and 8	$\frac{5}{\sqrt{2}}$	$\frac{1}{2\sqrt{2}}=0.354$	0.3555
3, 4, 9, and 10	$\frac{5\sqrt{10}}{2}$	$\frac{\sqrt{10}}{4}=0.790$	.1420
1, 2, 11, and 12	$\frac{5\sqrt{26}}{2}$	$\frac{\sqrt{26}}{4}=1.274$	.0429
Total			.5404

By taking $z=20$ feet and computing in the same manner, it is found that at the lower boundary of the compressible stratum and on the vertical center axis of the footing, p_z is 0.73 ton per square foot. The pressure is a maximum for points along the vertical center axis of the footing and the distribution of this maximum pressure is trapezoidal, case 5. At points removed from the vertical center axis there is also case 5 pressure distribution, but the value of p_z at any level is less than the value of p_z for the same level on the vertical center axis. For computations consider only the maximum pressures, which are along the vertical center axis.

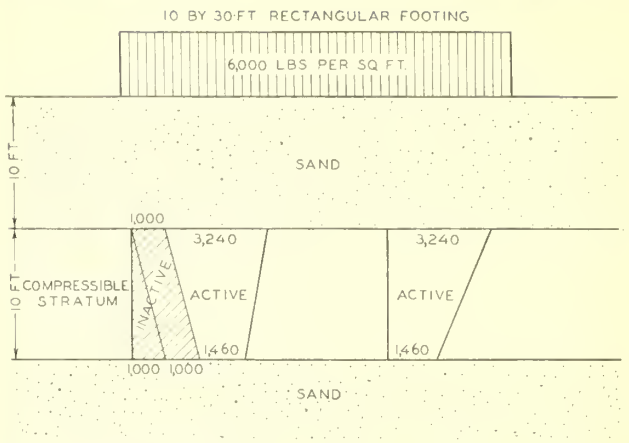

FIGURE 8.—DISTRIBUTION OF MAXIMUM PRESSURE UNDER RECTANGULAR FOOTING. SOIL LAYER CONTAINED BETWEEN TWO PERMEABLE SAND LAYERS.

The shaded trapezoidal area marked "inactive" in figure 8, denotes the pressure distribution resulting from the weight of earth material alone. The trapezoidal figure marked "active" denotes the distribution of maximum pressure in the compressible stratum that results from the uniform load on the footing. It is again assumed that consolidation of the stratum caused by its own weight and that of the sand is complete at the time the surface load is applied and hence all of this pressure has been transferred from water to soil and therefore cannot act in combination with the applied surface load in causing consolidation. The applied load producing the active trapezoidal pressure distribution produces further consolidation beyond that already effected by the weight of earth material.

When the surface load is applied, the pressure per square foot at the upper boundary of the compressible layer is 1,000+3,240=4,240 pounds or 2.12 tons. That at the lower boundary is 3,460 or 1.73 tons. From figure 2, these pressures correspond to voids ratios of 0.90 and 0.94, respectively, the average value of e being $0.92=e_2$. Prior to loading, the average value of e, as found in example 1, is 1.17. The ultimate settlement is therefore

$$Q=\frac{1.17-0.92}{1+1.17}\times 10=1.15 \text{ feet}=13.8 \text{ inches.}$$

The average voids ratio is reduced by the surface loading from 1.17 to 0.92. The average of these is 1.045 and the corresponding value of c from figure 3 is 0.0138.

Example 2 (continued).—Boundary condition A; the stratum is sandwiched between sand layers. For the time-consolidation computations, again apply equation (17) for boundary condition A (see fig. 8).

$$\text{For } q_1=25 \text{ percent, } t=\frac{N_1\left(\frac{D_0}{2}\right)}{1{,}400\ c}=\frac{0.12\times\left(\frac{4.61}{2}\right)^2}{1{,}400\times 0.0138}=0.033 \text{ year}=12 \text{ days.}$$

Similarly, $t=0.135$ year=49 days for $q_1=50$ percent and 0.325 year=119 days for $q_1=75$ percent.

These values are nearly equal to those computed for boundary condition A, example 1, and would be the

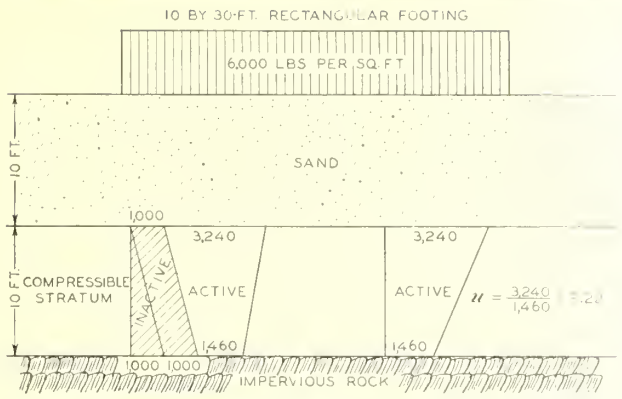

FIGURE 9.—DISTRIBUTION OF MAXIMUM PRESSURE UNDER RECTANGULAR FOOTING. SOIL LAYER OVERLAID BY SAND AND RESTING ON ROCK.

same were it not for small variations in c as the pressure is varied. For most practical purposes, when boundary condition A obtains, the time-consolidation relation can be considered independent of the amount of pressure and the type of pressure distribution.

Example 2 (continued).—Boundary condition B; the compressible stratum rests on rock and is overlaid with sand. When condition B obtains in this example (see fig. 9), use equations (24) and (26), together with tables 2 and 3. The total settlement, Q, is the same for both boundary conditions, A and B. The value for u (consider only the active pressure, fig. 9) is 2.22. Then J' is obtained from table 3 and is taken as 0.67. From table 2, when $q_1=q_3=q_5=25$ percent, $N_1=0.12$ and $N_3=0.03$, and from equation (26),

$$N_5=N_3+J'(N_1-N_3)=0.03+0.67(0.12-0.03)=0.09.$$

This value for N_5 is substituted in equation (24) and taking $c=0.0138$, then for $q_5=25$ percent,

$$t=\frac{N_5 D_0^2}{1{,}400\,c}=\frac{0.09\times(4.61)^2}{1{,}400\times 0.0138}=0.099 \text{ year}=36 \text{ days}.$$

For $q_1=q_3=q_5=50$ percent, N_1 (table 2)$=0.49$ and $N_3=0.24$.
Then $N_5=0.24+0.67\ (0.49-0.24)=0.41$ and

$$t=\frac{0.41\times(4.61)^2}{1{,}400\times 0.0138}=0.451 \text{ year}=165 \text{ days}.$$

Similarly for $q_5=75$ percent,

$$t=\frac{1.08\times(4.61)^2}{1{,}400\times 0.0138}=1.19 \text{ years}=435 \text{ days}.$$

Example 3.—Case 4, pressure distribution; type E loading; boundary condition B. It would seem that case 4 pressure distribution is a comparatively rare occurrence. Problems of this type may be encountered, however, when consolidation produced by the weight of earth material alone is incomplete at the time of load application at the ground surface. Figure 10 illustrates such a condition. If it is assumed that consolidation in the compressible stratum 5 feet thick resulting from the weight of sand above is only 50 percent completed, then this weight exerts an active pressure which is zero at the top and 1,000 pounds per square foot at the bottom of the compressible layer. The distribution of active pressure resulting from the weight of sand alone is, therefore, triangular. It is assumed that consolidation of the

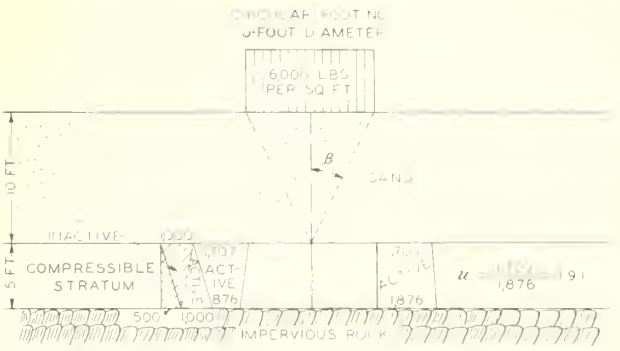

FIGURE 10.—DISTRIBUTION OF MAXIMUM PRESSURE UNDER CIRCULAR FOOTING. SOIL LAYER OVERLAID BY SAND AND RESTING ON ROCK.

compressible layer as produced by its own weight is complete at the time of loading.

Referring to figure 10, a load of 3 tons per square foot is applied over a rigid circular footing 10 feet in diameter. It is desired to determine the total settlement and the rate of settlement of the compressible layer which rests on rock and is covered with the sand layer 10 feet thick, 50 percent of the consolidation producible by the weight of sand being completed at the time of load application.

To compute the pressure, p_z, at a point on the vertical centerline 10 feet below the rigid footing, substitute values in equation (12). The cosine of angle β (fig. 10) is $\frac{10}{\sqrt{125}}=\frac{2}{\sqrt{5}}$ and p equals 3 tons per square foot. Then

$$p_z=p(1-\cos^3\beta)=3\left(1-\frac{8}{5\sqrt{5}}\right)=0.854 \text{ ton per square foot}.$$

At a point 15 feet below the footing and on the vertical center axis, $p_z=0.438$ ton per square foot. Therefore, the applied load produces a pressure of 1,707 pounds per square foot at the upper boundary and 876 pounds per square foot at the lower boundary of the compressible layer.

Prior to loading, e at the upper surface of the layer is 1.28, corresponding to a load of 1,000 pounds per square foot, and at the lower boundary it is 1.44, corresponding to a load of 500 pounds per square foot. The average is $1.36=e_1$. The voids ratios (see fig. 2) corresponding to pressures of $1{,}000+1{,}707$ pounds per square foot at the top and $500+1{,}876$ pounds per square foot at the bottom of the layer are found (fig. 2) to be 0.99 and 1.01, respectively. The average is $1.00=e_2$. The total settlement is, therefore,

$$Q=\left[\frac{1.36-1.00}{1+1.36}\right]\times 5=0.76 \text{ foot}.$$

It is important to note that prior to loading, soil at the top of the compressible stratum has reached equilibrium under the weight of sand (1,000 pounds per square foot) and that soil at the bottom of the stratum has not as yet been consolidated by the sand but has been consolidated by the weight of compressible soil, i. e., 500 pounds per square foot. Therefore, the voids ratio at the top of the layer prior to loading corresponds to a pressure of 1,000 and that at the bottom to a pressure of 500 pounds per square foot. After a very long period of time subsequent to loading, the soil at the top of the compressible layer will be in equilibrium with a pressure of 2,707 pounds per square foot and that at the bottom with a

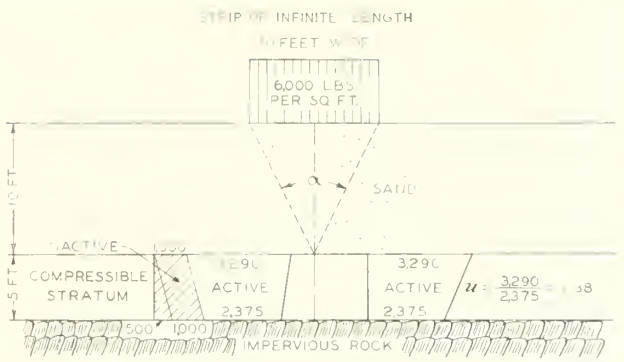

FIGURE 11.—DISTRIBUTION OF MAXIMUM PRESSURE UNDER STRIP OF INFINITE LENGTH. SOIL LAYER OVERLAID BY SAND AND RESTING ON ROCK.

pressure of 2,376 pounds per square foot. These pressures correspond to definite voids ratios, found experimentally (see fig. 2), and are used in computing Q.

When load is applied, the active pressure at the top of the layer is 1,707 and that at the bottom is 1,876 pounds per square foot. Hence $u=\frac{1,707}{1,876}=0.91$, a value between 0 and 1 (case 4). Apply equations (23) and (25) and use the values for N_1 and N_2 corresponding to different values for $q_1=q_2$, table 2. Values of J are found from table 4, and for $u=0.91$, $J=0.06$. For $q_1=q_2=25$ percent, $N_1=0.12$, $N_2=0.31$, and

$$N_4=N_1+J(N_2-N_1)=0.12-0.06(0.31-0.12)=0.13$$

$$D_0=\frac{1}{1+e_1}\cdot 5=\frac{1}{2.36}\times 5=2.12 \text{ feet.}$$

For $e=\frac{e_1+e_2}{2}=1.18$, $c=0.0106$ (fig. 3).

Then for $q_4=25$ percent, $t=\frac{N_4D_0^2}{1,400c}=\frac{0.13\times(2.12)^2}{1,400\times 0.0106}=0.039$ year $=14$ days.

Similarly, for $q_4=50$ percent, $t=\frac{0.50\times(2.12)^2}{1,400\times 0.0106}=0.15$ year $=55$ days,

and for $q_4=75$ percent, $t=\frac{1.19\times(2.12)^2}{1,400\times 0.0106}=0.36$ year $=132$ days. In 132 days the settlement is 75 percent of 0.76 foot or 6.8 inches.

Example 4.—Case 5 pressure distribution; type D loading; boundary condition B. All other conditions are the same as in example 3 except that in this case the rigid footing is a loaded strip 10 feet wide and of relatively great length instead of a circular bearing area and consolidation caused by weight of sand is complete prior to loading. Figure 11 shows a vertical cross section. To find p_z at the top of the compressible layer and at a point on the vertical center axis through the cross section, apply equation (11). The angle, α, is expressed in radians and $\sin\alpha=0.80$.

$$p_z=\frac{p}{\pi}(\alpha+\sin\alpha)=\frac{6,000}{3.14}(0.93+0.80)=3,290 \text{ pounds per square foot.}$$

At the lower boundary and on the centerline, $p_z=\frac{6,000}{3.14}(0.64+0.60)=2,375$ pounds per square foot. This is case 5 pressure distribution, and $u=1.38$. The average voids ratio prior to loading is the average of e corresponding to 1,000 and of e corresponding to 1,500 pounds per square foot and is $1.21=e_1$. Subsequent to loading the values of e at the top and bottom of the compressible stratum correspond to the equilibrium pressures, 4,290 and 3,875 pounds per square foot, respectively. From figure 2 these values of e are 0.90 and 0.92, the average being $0.91=e_2$. The total settlement is therefore

$$Q=\left[\frac{1.21-0.91}{1+1.21}\right]\times 5=0.68 \text{ foot or 8.2 inches.}$$

To compute the rate of consolidation, apply equations (24) and (26) and make use of tables 2 and 3. $J'=0.87$ for $u=1.38$ (by interpolation). For $e=\frac{e_1+e_2}{2}=1.06$, $c=0.0134$.

For $q_1=q_3=q_5=25$ percent, $t=\frac{N_5D_0^2}{1,400c}=\frac{0.11\times(2.26)^2}{1,400\times 0.0134}=0.03$ year $=11$ days.

For $q_5=50$ percent, $t=\frac{0.46\times(2.26)^2}{1,400\times 0.0134}=0.125$ year $=46$ days.

For $q_5=75$ percent, $t=\frac{1.14\times(2.26)^2}{1,400\times 0.0134}=0.310$ year $=113$ days.

In 113 days the strip settles 75 percent of 8.2 inches or 6.2 inches.

The preceding examples are intended to illustrate the application of the theory of soil consolidation. Part 2 contains the derivation of equations and the development of the basic theory.

PART 2.—DEVELOPMENT OF THEORY OF SOIL CONSOLIDATION

In 1885, J. Boussinesq [6] developed in considerable detail and usable form a solution of the general problem of determining the distribution of stresses and strains in a semi-infinite, elastic, isotropic solid, bounded by a plane and loaded by a single concentrated force at a point on that plane. The formulas of Boussinesq have had wide application in the development of the science of soil mechanics.

The distribution of stresses across any plane parallel to the plane surface is shown in figure 12. If a single concentrated vertical load, P, is applied at a point on the surface, then at a radial distance, R, from this point and at a depth, z, below the surface, the stresses at the point (x, y) are:

$$p_R = \frac{3P}{2\pi R^2}\frac{z}{R}, \text{ parallel to } R \tag{32}$$

$$p_x = \frac{3P}{2\pi R^2}\frac{zx^2}{R^3}, \text{ parallel to } x \text{ axis} \tag{33}$$

$$p_y = \frac{3P}{2\pi R^2}\frac{zy^2}{R^3}, \text{ parallel to } y \text{ axis} \tag{34}$$

$$p_z = \frac{3P}{2\pi R^2}\frac{z^3}{R^3}, \text{ parallel to } z \text{ axis} \tag{35}$$

The derivation of these equations may be found in almost any standard text dealing with the theory of elasticity.[7] These equations contain no elastic constants, however, and for this reason it has probably been assumed that the distribution of stresses does not depend upon the type of material. Cummings,[8] Krynine,[9] and others have discussed this point at length.

A complete analysis of the stress distribution including planes not parallel to the boundary involves 6 components of stress, 3 shears and 3 normal stresses, and the elastic constants appear in some of these equations. In the theory of consolidation of soils we are concerned mainly with the vertical normal stress, p_z. At a given depth, z, it is seen from equation (35) that p_z is greatest when R is smallest, that is, the maximum values for p_z are along the vertical centerline of the loaded area ($x=y=0$, fig. 12).

The value of p_z at any point is easily computed, knowing that $R^2=r^2+z^2=x^2+y^2+z^2$. If a combination of point loads is given, we may obtain the stresses at a point resulting from each load considered separately, then add these and obtain the total pressure at the point produced by the combination of loads. Gilboy [10] has shown that by dividing a uniformly loaded area into rectangular elements so that the longer side of any element is less than half the distance from the center of the area to the point, the computed value for p_z at the point will not deviate from the correct value by more than 6 percent. The accuracy is greater with further subdivision of the rectangular elements.

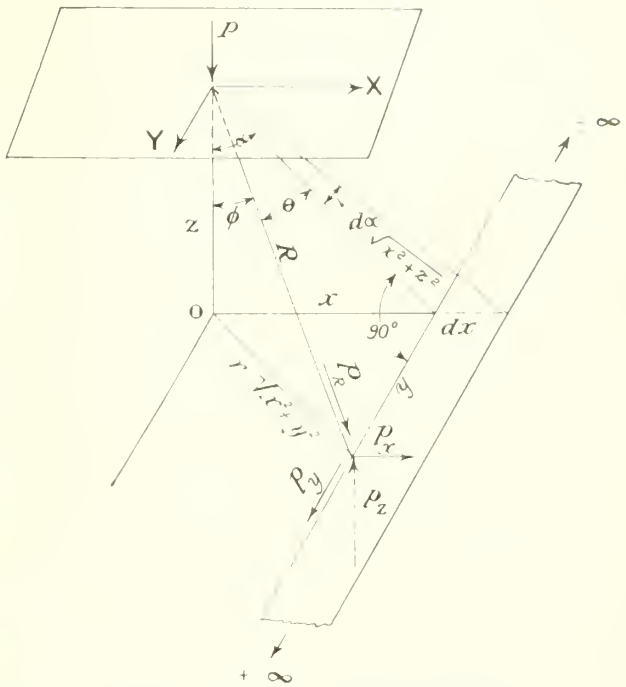

FIGURE 12.—DISTRIBUTION OF STRESSES ACROSS ANY PLANE PARALLEL TO THE PLANE SURFACE.

FORMULAS DEVELOPED FOR DETERMINING PRESSURE AT POINT BELOW LOAD

With reference to figure 12, if the expression for $p_z=\frac{3z^3P}{2\pi R^5}$ is integrated in the y direction from $y=-\infty$ to $y=+\infty$ and the resulting "line load" is multiplied by dx, we obtain the total load on a strip infinitely long and of width, dx, produced by the point load, P, at the boundary. In this integration, x and z are constant. Thus:

$$p_z = \frac{3Pz^3}{2\pi R^5} = \frac{3Pz^3}{2\pi(x^2+y^2+z^2)^{5/2}}$$

$$\int p_z = dx\frac{3Pz^3}{\pi}\int_0^{\infty}\left[\frac{x^2+y^2+z^2}{x^2+z^2}\right]^{-5/2}(x^2+z^2)^{-5/2}dy$$

This is easily integrated if we put $\frac{y}{(x^2+z^2)^{1/2}}=\tan\theta$ (see fig. 12). Then

$$\int p_z = (dx)\frac{3Pz^3}{\pi(x^2+z^2)^2}\int_{\theta=0}^{\theta=\pi/2}\cos^3\theta d\theta,$$

or

$$\int p_z = \frac{3Pz^3}{\pi(x^2+z^2)^2}\left[\sin\theta-\frac{\sin^3\theta}{3}\right]_0^{\pi/2}dx = \frac{2Pz^3}{\pi(x^2+z^2)^2}\,dx \quad (36)$$

If this expression is again integrated in the x direction, taking twice the value obtained between the limits, $x=0$ and $x=\infty$ (fig. 12), the result is equal to P, the applied force. Obviously, on any plane of infinite extent and at a finite distance below and parallel to the loading surface, the total distributed load is equal to the applied load at the surface. The mathematical result is in agreement with this self-evident fact.

[6] Application des Potentiels a l'Etude de l'Equilibre et du Movement des Solides Elastiques. Paris, 1885.

[7] For example, see chapter 11 of Theory of Elasticity, by S. Timoshenko

[8] Distribution of Stresses under a Foundation, by A. E. Cummings, Proceedings, American Society of Civil Engineers, vol. 61, no. 6, August 1935.

[9] Discussion of Cummings' paper by Clement C. Williams, D. P. Krynine, and L. C. Wilcoxen, Proceedings, American Society of Civil Engineers, vol. 68, no. 8, pt. I, October 1935.

[10] Earths and Foundations. Progress Report of Special Committee, Proceedings, American Society of Civil Engineers, vol. 59, no. 5, May 1933

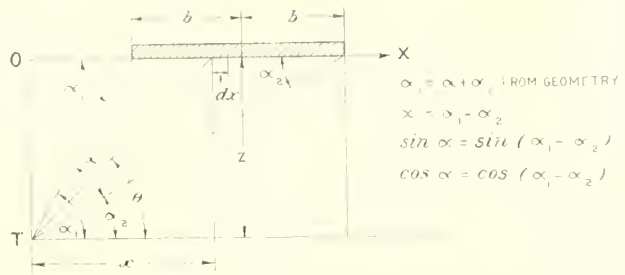

FIGURE 13.—VERTICAL CROSS SECTION OF A UNIFORMLY LOADED AREA OF INFINITE LENGTH AND OF WIDTH $2b$.

In equation (36), if P is taken as p, the vertical pressure per unit area, and if p is considered as concentrated at the center of a unit area, then the total vertical pressure on the strip at a distance below the surface is equal to $\frac{2pz^3}{\pi(x^2+z^2)^2}dx$. This is the total pressure, $\int p_z$, on a strip below the surface produced by a unit load on the surface. Conversely, the total load on an infinitely long strip at the surface and parallel to the y axis, produced by applying the unit load, p, over the length of this strip will cause a vertical pressure $\frac{2pz^3}{\pi(x^2+z^2)^2}dx$, at a point below the surface. Briefly, a point load at the surface produces a total load, $\frac{2pz^3}{\pi(x^2+z^2)^2}dx$ on a strip below the surface and conversely, a strip load of unit load per unit area on the surface produces a pressure, $\frac{2pz^3}{\pi(x^2+z^2)^2}dx$ at a point below the surface, action and reaction being equal. It will be noted that the integrations of the Boussinesq formulas are with reference to areas or points below the loaded area at the surface. At the surface, $z=0$ and there is nothing to integrate that has reference to the fundamental equation (35).

Figure 13 is a vertical cross section of a uniformly loaded area (p per unit of area) of infinite length and of width, $2b$. The load on a strip of infinitesimal width, dx, produces a pressure, p_z, at a point T equal to $\frac{2pz^3}{\pi(x^2+z^2)^2}dx$. If this expression is integrated between limits, there is obtained the value for p_z at T resulting from the total load over the whole area, infinite in length and of width, $2b$. The integration is more easily performed if we let $\frac{x}{z}=\cot\theta$ (fig. 13). Then at the point T,

$$p_z = \frac{2p}{\pi z}\int_{x-2b}^{x}\left(1+\frac{x^2}{z^2}\right)^{-2}dx = -\frac{2p}{\pi}\int_{\alpha_1}^{\alpha_2}(1+\cot^2\theta)^{-2}\csc^2\theta d\theta$$
$$=\frac{p}{\pi}[\alpha-\sin\alpha\cos(\alpha_1+\alpha_2)] \qquad (37)$$

If the point, T, is on the vertical centerline through the vertical cross section (fig. 13), then $\alpha_2=180°-\alpha_1$ and equation (37) reduces to

$$p_z=\frac{p}{\pi}(\alpha+\sin\alpha) \qquad (38)$$

For uniform pressure distribution (p per unit area) over a circular bearing area of radius r (see fig. 14) the

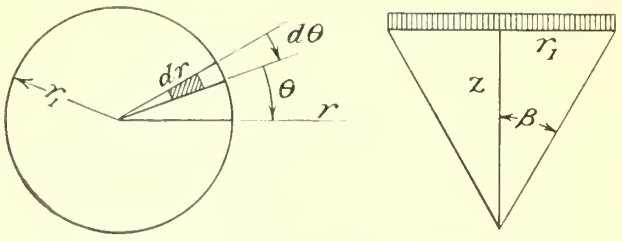

FIGURE 14.—CIRCULAR LOADED AREA.

pressure at a point below the loaded area and on the vertical centerline is,

$$p_z=\int_0^{r_1}\int_0^{2\pi}\frac{3p}{2\pi}\frac{z^3}{(z^2+r^2)^{5/2}}rdrd\theta=3pz^3\int_0^{r_1}\frac{rdr}{(r^2+z^2)^{5/2}}$$

Integrating and simplifying,

$$p_z=p\left[1-\frac{1}{\left(1+\frac{r_1^2}{z^2}\right)^{3/2}}\right] \text{ and since } \frac{r_1}{z}=\tan\beta$$

$$p_z=p(1-\cos^3\beta) \qquad (39)$$

MECHANICS OF SOIL CONSOLIDATION OUTLINED

Consider the flow or egress of water from a wet, compressible, soil stratum subjected to vertical loading. The soil is relatively impervious and is "sandwiched" between two permeable sand-gravel layers, L, L, figure 15. The movement of water saturating the stratum is in both the upward and downward directions, parallel to the vertical axis, ZZ, and directed toward the two permeable layers, L, L.

At any finite time, t, after load application, that part of the total pressure carried by the water, p_w, is considered as the "hydrodynamic excess." [11] The remainder of the load is carried by the solid material, the "soil skeleton." At the outset of loading ($t=0$) all of the pressure is carried by water if the compressible layer is saturated. At any subsequent time, the hydrodynamic excess, p_w, is zero at the surfaces of contact of the soil stratum and the permeable layers, but elsewhere it is a finite quantity, varying with the depth, z. The upper and lower faces, AB and CD of the elementary prism, ABCD, are each of unit area. The thickness of the elementary prism is Δz (fig. 15).

We may then define the following expressions:

p=the total pressure per unit cross-sectional area. It includes the pressure on the soil skeleton and the pressure, p_w, carried by water alone.

p_w=the pressure per unit cross-sectional area that is carried by water alone.

$p-p_w$=the pressure per unit cross-sectional area that is carried only by the soil particles, i. e., by the soil skeleton.

a=the rate of change of the voids ratio with respect to $(p-p_w)$ and is called the coefficient of compressibility.

k_0=the reduced coefficient of permeability; see equation (4), Part 1.

$\frac{dp_w}{dz}$=the rate of change of the hydraulic pressure, p_w, with respect to the depth, z. It is the hydraulic pressure gradient.

$\frac{d^2p_w}{dz^2}$=the rate of change of $\frac{dp_w}{dz}$ with respect to z and

[11] Terzaghi, K., Erdbaumechanik. Franz Deuticke, Vienna, 1925.

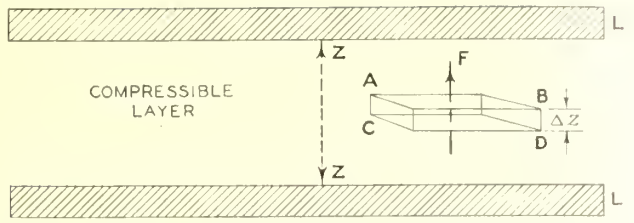

FIGURE 15.—ELEMENTARY PRISM OF COMPRESSIBLE MATERIAL BETWEEN TWO PERMEABLE LAYERS. F INDICATES DIRECTION OF FLOW.

is taken as constant through the very small vertical distance, Δz.

$\Delta z\frac{d^2p_w}{dz^2}$ = the total change of $\frac{dp_w}{dz}$ within the interval, Δz.

$k_0\frac{d^2p_w}{dz^2}\Delta z$ = the rate at which the elementary prism loses water. It represents the difference between the quantity of water flowing from the upper face, AB, in unit time and the quantity flowing into the prism through the face, CD, in unit time. It is therefore the rate of volume reduction of the elementary prism.

$\frac{dp_w}{dt}=-\frac{d(p-p_w)}{dt}$ = the rate of change of $p-p_w$ with respect to time.

$a\frac{dp_w}{dt}$ = the rate of change in voids ratio with respect to time.

$a\frac{dp_w}{dt}\Delta z$ = the rate of change in volume of the elementary prism with respect to time.

The expressions $\frac{dp_w}{dz}$, $\frac{d^2p_w}{dz^2}$, and $\frac{dp_w}{dt}$ are considered as the partial derivatives. The faces, AB and CD, of the elementary prism are both of unit horizontal cross-sectional area and are parallel to each other.

The amount of water lost by the elementary prism in unit time is $k_0\frac{d^2p_w}{dz^2}\Delta z$, which is the difference between the amount of water flowing out from the face AB and that flowing into the prism through the face CD. But the rate of loss of water (which is the rate of reduction in volume) by the elementary prism is also equal to $a\frac{dp_w}{dt}\Delta z$. Then $k_0\frac{d^2p_w}{dz^2}\Delta z=a\frac{dp_w}{dt}\Delta z$. Divide both sides of this expression by a and write c for $\frac{k_0}{a}$. Then, since Δz cancels,

$$c\frac{d^2p_w}{dz^2}=\frac{dp_w}{dt} \qquad (40)$$

This is analogous to the development of the equation for the flow of heat. A more complete derivation of equation (40) has been published.[12]

TIME-CONSOLIDATION RELATIONSHIPS FOR UNIFORM PRESSURE DISTRIBUTION DERIVED

A solution of equation (40) is

$$p_w=e^{-cK^2t}(A\cos Kz+B\sin Kz) \qquad (41)$$

where A, B, and K are constants.

[12] Mathematical Theory of the Process of Consolidation of Mud Deposits, by Alberto Ortenblad. Journal of Mathematics and Physics, vol. 9, no. 2

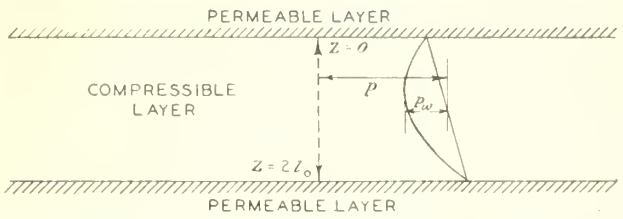

FIGURE 16.—PRESSURE DISTRIBUTION IN A COMPRESSIBLE LAYER BETWEEN TWO PERMEABLE LAYERS. AT ANY LEVEL, Z, p_w = THE PRESSURE TAKEN BY WATER. p = THE TOTAL PRESSURE. $p-p_w$ = THE PRESSURE ON THE SOIL SKELETON.

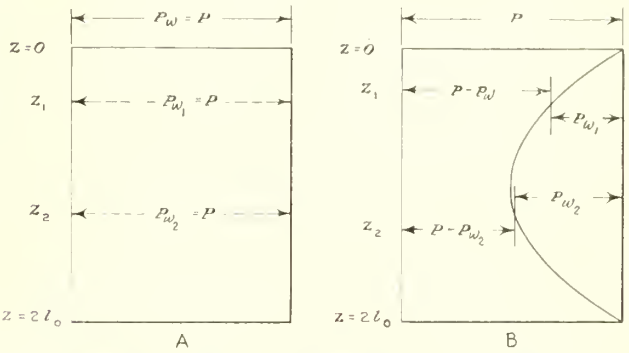

FIGURE 17.—A, PRESSURE CONDITIONS IN COMPRESSIBLE LAYER. AT THE INSTANT LOAD IS APPLIED ($t=0$), $p_w=p$ AT ALL LEVELS. B, PRESSURE CONDITIONS IN COMPRESSIBLE LAYER SUBSEQUENT TO LOAD APPLICATION. WHEN $t>0$, $p_w=0$ AT $z=0$ AND AT $z=2\,l_0$. $p_w<p$ AT ALL LEVELS.

In figure 16, the reduced thickness (the thickness the compressible stratum would have were there no voids between the particles) of the soil layer is $2l_0$ where l_0 is the maximum distance that water travels if the layer has a thickness, d_0. That is, $d_0=2l_0$. Water saturates the soil. Apply load. At the very instant that load is applied, as noted before, the water in the soil takes all the pressure (see fig. 17–A). That is, at $t=0$, the total pressure, p, is equal to p_w at any depth, z (see fig. 17–A). At any other time after loading, p_w is less than the total pressure, some of which has been transferred to the soil skeleton. The condition, subsequent to applying load, is represented by figure 17–B. At any level, z, a part of the load, p_w, designated as the "hydrodynamic excess," is carried by water. The load carried by the soil skeleton at depth, z, is the total pressure minus p_w.

At any time subsequent to $t=0$, p_w is equal to 0 when $z=0$. The water escapes instantaneously at $z=0$.

The same condition prevails at the lower boundary, that is at $z=2l_0$, $p_w=0$ if t is greater than 0. At $t=0$, p_w is a function of z.

For present purposes, assume that the total pressure, p, (that on solid and that on water taken together) is uniform and constant throughout the depth, z.

We now consider equation (41) under the following boundary conditions.

$$p_w=(A\cos Kz+B\sin Kz)e^{-cK^2t} \qquad (41)$$

Condition 1: $p_w=0$ when $z=0$ and $t>0$.

The sine of 0 is 0 and $\sin Kz=0$.

Therefore, since $p_w=0$ and $B\sin Kz=0$, it follows that $A\cos Kz$ must also $=0$. But $\cos 0=1$ and $A\cos 0=A$. Then A must equal zero. So condition 1 reduces equation (41) to

$$p_w=(B\sin Kz)e^{-cK^2t} \qquad (42)$$

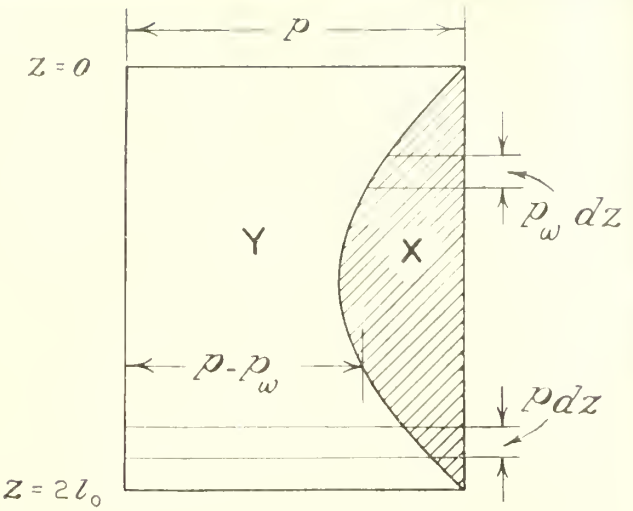

FIGURE 18. FRACTIONS OF LOAD CARRIED BY SOIL SKELETON AND BY WATER.

Condition 2: When $z=2l_0$ and $p_w=0$, then $\sin Kz = \sin 2Kl_0=0$.

Hence Kz must be an integral multiple of π.

$z=2l_0$ and $Kz=2Kl_0$ for condition 2, and $\sin Kz=0$ when $Kz=2Kl_0=\pi, 2\pi, 3\pi \ldots n\pi$, where n is any integer from 1 to infinity.

Since $2Kl_0=n\pi$, $K=\frac{\pi}{2l_0}=\frac{2\pi}{2l_0}=\frac{3\pi}{2l_0}=\frac{n\pi}{2l_0}$.

The expression, e^{-cK^2t}, becomes $e^{-\frac{n^2\pi^2ct}{4l_0^2}}$. Substitute for K the successive values, $\frac{\pi}{2l_0}, \frac{2\pi}{2l_0}, \frac{n\pi}{2l_0}$, etc. in equation (42), add and obtain:

$$p_w=a_1e^{-\frac{\pi^2ct}{4l_0^2}}\sin\frac{\pi z}{2l_0}+a_2e^{-\frac{4\pi^2ct}{4l_0^2}}\sin\frac{2\pi z}{2l_0}+\cdots+a_ne^{-\frac{n^2\pi^2ct}{4l_0^2}}\sin\frac{n\pi z}{2l_0} \quad \ldots(43)$$

When $t=0$, equation (43) becomes:

$$p_w-p=a_1\sin\frac{\pi z}{2l_0}+a_2\sin\frac{2\pi z}{2l_0}+a_3\sin\frac{3\pi z}{2l_0}+\cdots+a_n\sin\frac{n\pi z}{2l_0} \quad \ldots(44)$$

This is true for all values of z between 0 and $2l_0$. To find a_1, multiply through by $\sin\frac{\pi z}{2l_0}$ and integrate all terms between the limits, 0 and $2l_0$. Then, as usually happens in the case of a Fourier series, all terms on the right, except the first, vanish, and since p_w=a constant $=p$ when $t=0$,

$$\text{then } p\int_0^{2l_0}\sin\frac{\pi z}{2l_0}dz=a_1\int_0^{2l_0}\sin^2\frac{\pi z}{2l_0}dz \quad \ldots(45)$$

$$a_1=\frac{4p}{\pi},\ a_2=0,\ a_3=\frac{4p}{3\pi},\ a_n=\frac{4p}{n\pi},$$

where n is odd.

Substituting the values of $a_1, a_2, a_3, \cdots a_n$ in equation (43), there results

$$p_w=\frac{4p}{\pi}e^{-\frac{\pi^2ct}{4l_0^2}}\sin\frac{\pi z}{2l_0}+\frac{4p}{3\pi}e^{-\frac{9\pi^2ct}{4l_0^2}}\sin\frac{3\pi z}{2l_0}+\cdots+\frac{4p}{(2n+1)\pi}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}\sin\frac{(2n+1)\pi z}{2l_0} \quad \ldots(46)$$

Where n is 0, 1, 2, 3, etc.

Multiply equation (46) through by dz and integrate between the limits, $z=0$ and $z=2l_0$. We have:

$$\int_0^{2l_0}p_w dz=\frac{4p}{\pi}e^{-\frac{\pi^2ct}{4l_0^2}}\int_0^{2l_0}\sin\frac{\pi z}{2l_0}dz+\frac{4p}{3\pi}e^{-\frac{9\pi^2ct}{4l_0^2}}\int_0^{2l_0}\sin\frac{3\pi z}{2l_0}dz+\cdots+\frac{4p}{(2n+1)\pi}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}\int_0^{2l_0}\sin\frac{(2n+1)\pi z}{2l_0}dz \quad \ldots(47)$$

$$\text{or } \int_0^{2l_0}p_w dz=-\frac{8l_0p}{\pi^2}e^{-\frac{\pi^2ct}{4l_0^2}}\left[\cos\frac{\pi z}{2l_0}\right]_0^{2l_0}-\frac{8pl_0}{9\pi^2}e^{-\frac{9\pi^2ct}{4l_0^2}}\left[\cos\frac{3\pi z}{2l_0}\right]_0^{2l_0}\cdots-\frac{8pl_0}{(2n+1)^2\pi^2}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}\left[\cos\frac{(2n+1)\pi z}{2l_0}\right]_0^{2l_0}$$

$$=\frac{16pl_0}{\pi^2}\sum_{n=0}^{n=\infty}\frac{1}{(2n+1)^2}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}$$

This is the total p_w at any time greater than zero.

The total p_w at $t=0=\int_0^{2l_0}p\,dz=p[z]_0^{2l_0}=2l_0p$

since $p_w=p$ when $t=o$.

$\int_0^{2l_0}p_w dz$=total p_w at any subsequent time, t.

$1-\frac{\int_0^{2l_0}p_w dz}{2l_0p}=q_1$, the percentage of consolidation at time, t.

That is

$$q_1=1-\frac{16pl_0}{\pi^2}\left(e^{-N}+\frac{e^{-9N}}{9}+\frac{e^{-25N}}{25}+\frac{e^{-49N}}{49}+\cdots\right)\Big/2pl_0$$

$$\text{or } q_1=1-8/\pi^2\left(e^{-N}+\frac{e^{-9N}}{9}+\frac{e^{-25N}}{25}+\frac{e^{-49N}}{49}+\cdots\right) \quad (48)$$

where $N=\frac{\pi^2ct}{4l_0^2}$

Figure 18 illustrates diagrammatically the load fractions carried by the clay skeleton and the water.

In figure 19, X = load fraction carried by water, and Y = load fraction carried by soil.

The total $p_w=X=\int_0^{2l_0}p_w\,dz$ and $X+Y=\int_0^{2l_0}p\,dz$=the total pressure.

$$q_1=1-\frac{\int_0^{2l_0}p_w\,dz}{\int_0^{2l_0}p\,dz}=\frac{Y}{X+Y}, \text{ when } t=t_1>0$$

$$q_1=1-8/\pi^2\left(e^{-\frac{\pi^2ct}{4l_0^2}}+9e^{-\frac{9\pi^2ct}{4l_0^2}}+\cdots\frac{1}{(2n+1)^2}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}\right)$$

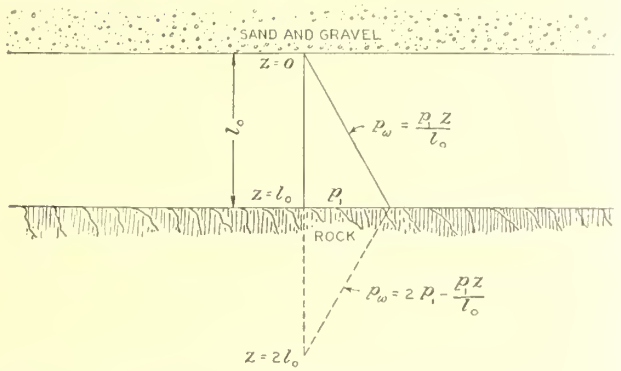

FIGURE 19.—TRIANGULAR PRESSURE DISTRIBUTION. THE LOWER TRIANGLE IS THE MIRROR IMAGE OF THE UPPER ONE.

DERIVATION OF TIME-CONSOLIDATION RELATIONSHIP FOR TRIANGULAR PRESSURE DISTRIBUTION, PRESSURE INCREASING WITH DEPTH

For a triangular pressure distribution (see fig. 19) the two boundary conditions,

$$\text{Condition 1: } p_w=0 \text{ at } z=0$$

$$\text{and condition 2: } \frac{dp_w}{dz}=0 \text{ at } z=l_0$$

must satisfy equation (41),

$$p_w=e^{-K^2ct}\ (A\cos Kz+B\sin Kz).$$

From condition 1, this equation reduces to equation (42) as before and

$$p_w=e^{-K^2ct}\ (B\sin Kz) \qquad (42)$$

For the second condition, at $z=l_0$,

$$\frac{dp_w}{dz}=e^{-K^2ct}\,BK\cos Kz=0=e^{-K^2ct}\,BK\cos Kl_0.$$

But

$$\cos Kl_0=0 \text{ when } Kl_0=\frac{m\pi}{2}$$

where m is any odd integer. That is,

$$K=\frac{\pi}{2l_0},\frac{3\pi}{2l_0},\frac{5\pi}{2l_0},\ldots,\frac{m\pi}{2l_0}.$$

Substitute these values of K successively in (42) and add the results together; we thus obtain

$$p_w=a_1e^{-\frac{\pi^2ct}{4l_0^2}}\sin\frac{\pi z}{2l_0}+a_2e^{-\frac{9\pi^2ct}{4l_0^2}}\sin\frac{3\pi z}{2l_0}+\cdots$$

to infinity (49)

At $t=0$, equation (49) becomes

$$p_w=a_1\sin\frac{\pi z}{2l_0}+a_2\sin\frac{3\pi z}{2l_0}+\cdots+a_m\sin\frac{m\pi z}{2l_0} \qquad (50)$$

where m is 1, 3, 5, etc., i. e., any odd, positive integer.

The coefficient, a_m, is evaluated in the usual manner. At $t=0$, $p_w=f(z)=\frac{p_1z}{l_0}$ between the limits $z=0$ and $z=l_0$. At $t=0$, between the limits $z=l_0$ and $z=2l_0$, $p_w=f_1(z)=2p_1-\frac{p_1z}{l_0}$.

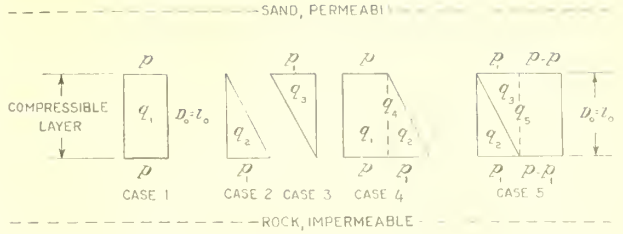

FIGURE 20.—TYPES OF PRESSURE DISTRIBUTION IN A COMPRESSIBLE LAYER.

Then

$$a_m=\frac{1}{l_0}\left[\int_0^{l_0}\frac{p_1z}{l_0}\sin\frac{m\pi z}{2l_0}\,dz+\int_{l_0}^{2l_0}\left(2p_1-\frac{p_1z}{l_0}\right)\sin\frac{m\pi z}{2l_0}\,dz\right]$$

Where m is any odd, positive integer.

Integrating, $a_m=\frac{8p_1}{m^2\pi^2}$ for $m=1, 5, 9, 13, 17$, etc.,

and

$$a_m=-\frac{8p_1}{m^2\pi^2} \text{ for } m=3, 7, 11, 15, 19, \text{ etc.}$$

By substitution equation (49) becomes

$$p_w=\frac{8p_1}{\pi^2}e^{-\frac{\pi^2ct}{4l_0^2}}\sin\frac{\pi z}{2l_0}-\frac{8p_1}{9\pi^2}e^{-\frac{9\pi^2ct}{4l_0^2}}\sin\frac{3\pi z}{2l_0}\pm\cdots \qquad (51)$$

$$\text{Then } \int_0^{l_0}p_w\,dz=\frac{8p_1}{\pi^2}\sum_{m=1}^{m=\infty}\frac{(-1)^{\frac{m-1}{2}}}{m^2}e^{-\frac{m^2\pi^2ct}{4l_0^2}}\int_0^{l_0}\sin\frac{m\pi z}{2l_0}\,dz$$

$$=\frac{16p_1l_0}{\pi^3}e^{-\frac{\pi^2ct}{4l_0^2}}-\frac{16p_1l_0}{27\pi^3}e^{-\frac{9\pi^2ct}{4l_0^2}}\pm\cdots\frac{16p_1l_0^2}{(2n+1)^3\pi^3}e^{-\frac{(2n+1)^2\pi^2ct}{4l_0^2}}$$

Where $n=0, 1, 2, 3$, etc.

At $t=0$, $f(z)=\frac{p_1z}{l_0}$ and $\int_0^{l_0}f(z)\,dz=\frac{p_1l_0}{2}$

$$q_2=1-\frac{\int_0^{l_0}p_w\,dz}{\int_0^{l_0}f(z)\,dz}$$

$$=1-\frac{32}{\pi^3}\left(e^{-\frac{\pi^2ct}{4l_0^2}}-\frac{1}{27}e^{-\frac{9\pi^2ct}{4l_0^2}}+\frac{1}{125}e^{-\frac{25\pi^2ct}{4l_0^2}}-+\cdots\right)$$

or writing $N=\frac{\pi^2ct}{4l_0^2}$,

$$q_2=1-\frac{32}{\pi^3}\left(e^{-N}-\frac{1}{27}e^{-9N}+\frac{1}{125}e^{-25N}-+\cdots\right) \qquad (52)$$

FORMULAS APPLICABLE TO OTHER TYPES OF PRESSURE DISTRIBUTION DEVELOPED

General relations.—Figure 20 illustrates five different types of pressure distribution. In this figure the conditions illustrated are for a compressible stratum overlaid with sand and resting on impervious rock.

The percentages of consolidation for the five types of pressure distribution shown in figure 21 are expressed as follows:

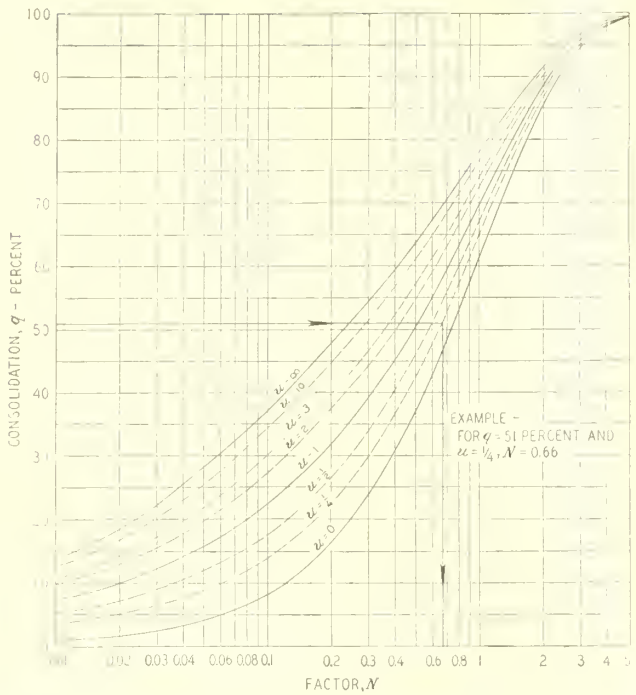

FIGURE 21.—VALUES OF q AND N FOR DIFFERENT VALUES OF u.

Case 1.—Uniform or rectangular pressure distribution.

$$q_1=1-\frac{8}{\pi^2}\left(e^{-N}+\frac{1}{9}e^{-9N}+\frac{1}{25}e^{-25N}+\ldots\right)$$

Case 2.—Triangular pressure distribution, pressure=0 at upper surface.

$$q_2=1-\frac{32}{\pi^3}\left(e^{-N}-\frac{1}{27}e^{-9N}+\frac{1}{125}e^{-25N}-,+\ldots\right)$$

Case 3.—Triangular pressure distribution, pressure=0 at lower surface.

$$q_3=\frac{p\left(q_1-\frac{q_2}{2}\right)}{\frac{p}{2}}=2q_1-q_2$$

Case 4.—Trapezoidal pressure distribution, pressure increasing with depth.

$$q_4=\frac{pq_1+q_2\frac{p_1}{2}}{p+\frac{p_1}{2}}$$

Case 5.—Trapezoidal pressure distribution, pressure decreasing with depth.

$$q_5=\frac{pq_1-\frac{p_1}{2}q_2}{p-\frac{p_1}{2}}=\frac{(p-p_1)q_1+\frac{p_1}{2}q_3}{p-\frac{p_1}{2}}$$

If the compressible layer is in contact with permeable sand layers above and below, then equation (48) holds for all of the five types of pressure distribution shown in figure 20 and interpolation formulas are not needed in this comparatively simple case for, if q_1, the percentage consolidation, is plotted against the time, the curves for the different pressure distributions will theoretically coincide for the same soil. The ultimate amount of consolidation may vary with the type of pressure distribution but the percentage of the ultimate consolidation attained at the end of a definite period of time is theoretically independent of the pressure or its manner of distribution if the relatively impervious stratum is confined between two relatively permeable layers, and the rate at which q_1 increases depends solely on the permeability, compressibility, and thickness of the clay layer.

The expressions for cases 4 and 5, trapezoidal pressure distributions, are combinations of equations (48) and (52) and are obtained by integration according to the procedure followed in deriving equation (52). The expressions are, case 4,

$$q_4=\frac{pq_1+\frac{p_1}{2}q_2}{p+\frac{p_1}{2}} \qquad (53)$$

and case 5,

$$q_5=\frac{pq_1-\frac{p_1}{2}q_2}{p-\frac{p_1}{2}} \qquad (54)$$

Case 3, triangular pressure distribution, u being infinite, is a special case of equation (54). For case 3, $p=p_1$ and hence from equation (54),

$$q_3=2q_1-q_2 \qquad (55)$$

When the expressions on the right of equations (48) and (52) are substituted for q_1 and q_2, equation (55) is written

$$q_3=2-\frac{16}{\pi^2}\left[e^{-N}+\frac{1}{9}e^{-9N}+\frac{1}{25}e^{-25N}+\ldots\right]$$
$$-1+\frac{32}{\pi^3}\left[e^{-N}-\frac{1}{27}e^{-9N}+\frac{1}{125}e^{-25N}-,+\ldots\right] \qquad (56)$$

Formulas for trapezoidal pressure distributions.—With reference to figure 20, both q_4 and q_5 approach q_1 as a limit as p_1 approaches the limit zero. This is also evident from equations (53) and (54). Note also that $p+\frac{p_1}{2}$ is the area of the trapezoid, case 4, when $l_0=1$ and $p-\frac{p_1}{2}$ is the area of the trapezoid, case 5, for $l_0=1$.

For the condition, $q_1=q_2=q_4$, the numerical value, N_4, will be between that for N_1 and N_2, that is,

$$N_4=N_1+J(N_2-N_1).$$

This is equation (25), Part 1. Then

$$(1-q_1)=(1-q_2)=(1-q_4) \text{ and } \log_e(1-q_1)$$
$$=\log_e(1-q_2)=\log_e(1-q_4)=\log_e R.$$

Neglecting all terms in e except the first in equations (48), (52), and (53),

$$\log_e(1-q_1)=\log_e\frac{8}{\pi^2}-N_1=\log_e R \qquad (57)$$

$$\log_e (1-q_2)=\log_e \frac{32}{\pi^3}-N_2=\log_e R \quad \text{(58)}$$

$$\log_e (1-q_4)=\log_e\left[\frac{\frac{8}{\pi^2}p+\frac{16}{\pi^3}p_1}{p+\frac{p_1}{2}}\right]-N_4=\log_e R \quad \text{(59)}$$

where $\log_e$ denotes the natural logarithm. Equations (57), (58), and (59) are easily solved for N_1, N_2, and N_4 and the values are substituted in the expression,

$$J=\frac{N_4-N_1}{N_2-N_1} \quad \text{(60)}$$

which is equation (25) solved for J. Thus

$$J=\frac{\log\left[\frac{2p\pi+4p_1}{\pi(2p+p_1)}\right]}{\log\frac{4}{\pi}} \quad \text{(61)}$$

Since for case 4, $u=\frac{p}{p+p_1}$ or $p=\frac{p_1 u}{1-u}$, then by substitution in equation (61) one obtains

$$J=\frac{\log\left[\frac{2(\pi-2)u+4}{\pi(1+u)}\right]}{\log\frac{4}{\pi}} \quad \text{(62)}$$

which is equation (21), Part 1.

For the condition, $q_1=q_3=q_5$, one may write

$$J'=\frac{N_5-N_3}{N_1-N_3} \quad \text{(63)}$$

Equation (63) is equation (26), Part 1, solved for J'. Furthermore, $1-q_1=1-q_3=1-2q_1+q_2$, from equation (55), $=1-q_5$. By dropping all terms in e except the first,

$$\log_e (1-q_1)=\log_e \frac{8}{\pi^2}-N_1=\log_e R \text{ as before} \quad \text{(57)}$$

and

$$\log_e (1-q_3)=\log_e (1-2q_1+q_2)-N_3$$

$$=\log_e\left[\frac{16}{\pi^2}-\frac{32}{\pi^3}\right]-N_3=\log_e R \quad \text{(64)}$$

In equation (54), $2q_1-q_3$ may be substituted for q_2 to obtain

$$q_5=\frac{(p-p_1)q_1+\frac{p_1}{2}q_3}{p-\frac{p_1}{2}} \quad \text{(65)}$$

Then

$$\log_e (1-q_5)=\log_e\left[\frac{(p-p_1)\frac{8}{\pi^2}+p_1\left(\frac{8}{\pi^2}-\frac{16}{\pi^3}\right)}{p-\frac{p_1}{2}}\right]-N_5 \quad \text{(66)}$$

After substituting the values obtained by solving equations (57), (64), and (66) for N_1, N_3, and N_5 in equation (63) and substituting $\frac{p_1 u}{u-1}$ for p, the expression obtained for J' is

$$J'=\frac{\log\left[\frac{u(\pi-2)+2}{(\pi-2)(u+1)}\right]}{\log\left[\frac{\pi}{2(\pi-2)}\right]} \quad \text{(67)}$$

This is equation (22) of Part 1.

The expression for J was derived by Gilboy.[13] The expression for J' was given by Kimball.[14] Neither Gilboy nor Kimball indicated the derivations and both have given negative signs for equations (62) and (67). It would of course be impossible to obtain table 3 of Part 1 of this paper with negative values for J and J'. Since only the first terms in e were used in their derivations, the expressions for J and J' are of course only approximate.

Figure 21 shows curves obtained by plotting values for N on log scale as abscissae against corresponding values for q plotted as ordinates. The different curves are for 8 particular values assigned to u, these being $u=0$, ¼, ½, 1, 2, 3, 10, and ∞. For intermediate values of u, values of q and N may be found by interpolation.

CHARACTERISTICS OF THEORETICAL TIME-CONSOLIDATION CURVE DISCUSSED

Equation (48) is one of a smooth curve having several important characteristics, one of which is illustrated in figure 22–A and 22–B. Figure 22–A is the curve obtained by taking $t=N$, that is, $\frac{\pi^2 c}{4l_0^2}$ as unity in equation (48). Let q_1 be the percentage consolidation at the end of a definite time interval, t, and t_1 the time required to produce half this percentage of consolidation, i.e., $\frac{1}{2}$ q_1. For $q_1=80$ percent, figure 22–A, $t=1.4$ minutes and the time, t_1, required to effect $\frac{1}{2}$ of 80 percent consolidation is seen to be 0.3 minute from the time consolidation curve, figure 22–A. The ratios, $\frac{t}{t_1}$ are plotted against q_1 in figure 22–B. Within the range $q_1=0$ to $q_1=70$ percent, the ratio, $\frac{t}{t_1}$ is fairly constant and is approximately equal to 4. This indicates that within this range the time-consolidation curve is approximately a parabola. For values of q_1 greater than 70 percent, the ratios, $\frac{t}{t_1}$, increase. An ordinate value of $q_1=90$ percent on the theoretical curve, figure 22–A, corresponds to a value of $\frac{t}{t_1}=5.27$, figure 22–B. The value, $t=2.1$ minutes, corresponds to the value, $\frac{t}{t_1}=5.27$.

Since the relation between q_1 and N_1 is fixed, the ratio, $\frac{t}{t_1}=5.27$ for $q_1=90$ percent is invariant for all values assigned to $\frac{\pi^2 c}{4l_0^2}$.

As a rule the time-compression curves for relatively impermeable soils have very nearly the same shape as a theoretical curve up to a point which, on the theoretical curve, has an ordinate of 90 percent. Beyond this point the two curves diverge appreciably. Since an

[13] See equation (31) in "Earths and Foundations," Proceedings, American Society of Civil Engineers, vol. 59, no. 5, May 1933.

[14] Proceedings, American Society of Civil Engineers, vol. 59, no. 6, August 1933.

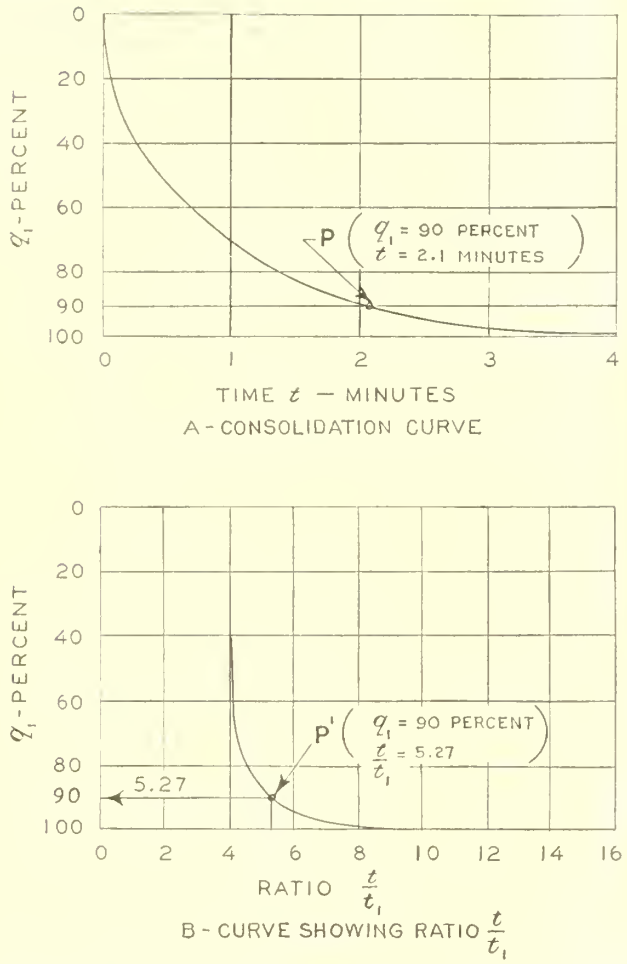

FIGURE 22.- RELATIONS BETWEEN PERCENTAGE OF CONSOLIDATION AND TIME, WHEN $t=N$.

ordinate of 90 percent on the theoretical curve corresponds to a definite value of $\frac{t}{t_1}=5.27$, it is easy to compute the coefficient of consolidation of a soil by adapting the actual curve to the theoretical one according to the following procedure:

1. Plot the laboratory time-consolidation curve, corresponding to figure 22–A.

2. From the time-consolidation curve plot the $\frac{t}{t_1}$ ratios corresponding to definite percentages of consolidation.

3. From the $\frac{t}{t_1}$, q_1 curve find the value of q_1 corresponding to $\frac{t}{t_1}=5.27$.

4. From the time-consolidation curve find the value of t corresponding to the value of q_1 for $\frac{t}{t_1}=5.27$. Call this particular value t_2.

5. Substitute the value for t_2 thus found in the expression, $\frac{\pi^2 c}{4l_0^2}t_2=2.1$, and solve for c,

$$c=\frac{2.1\times 4l_0^2}{\pi^2 t_2}=\frac{0.85l_0^2}{t_2} \quad \text{----------(68)}$$

In testing with the Terzaghi compression device, the sample is confined between two permeable layers and hence $l_0=\frac{1}{2}d_0$. In this case,

$$c=\frac{0.85d_0^2}{4t_2} \quad \text{----------(69)}$$

This is another method of computing c. [See equation (8).]

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF FEBRUARY 28,1937

STATE	APPORTIONMENT[1]	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,872,980	$ 51,600	$ 25,800	9.0	$ 911,981	$ 455,990	36.9	$ 801,590	$ 400,780	46.5	$ 6,990,410
Arizona	5,394,661	1,930,893	1,508,964	111.1	1,449,258	1,087,452	51.8	288,184	204,801	6.6	2,593,444
Arkansas	6,463,681	-	-	-	1,414,897	1,413,572	66.4	1,936,157	1,934,928	127.8	3,115,181
California	14,366,891	5,742,065	3,295,486	148.5	8,617,998	4,920,707	219.9	2,197,988	1,128,852	42.8	5,021,846
Colorado	6,911,198	3,800,010	2,030,922	150.4	2,999,222	1,667,481	103.6	935,170	523,516	49.4	2,689,278
Connecticut	2,388,339	777,897	388,949	14.2	705,928	350,556	8.7	-	-	-	1,648,834
Delaware	1,843,750	454,124	223,048	21.9	430,080	214,830	12.9	257,325	128,658	7.0	1,277,214
Florida	5,020,323	832,414	410,828	27.5	1,901,255	950,623	61.8	409,870	204,935	.7	3,453,937
Georgia	9,569,722	1,145,317	535,720	86.2	2,552,647	1,276,308	133.0	791,880	395,940	41.9	7,361,754
Idaho	4,635,991	2,698,759	1,570,730	256.6	866,009	518,151	56.1	751,776	455,870	39.9	2,091,240
Illinois	15,564,720	8,317,693	4,151,826	134.5	4,575,896	2,253,474	122.6	4,302,725	2,128,210	113.2	7,031,210
Indiana	9,333,269	5,402,340	2,690,678	179.1	2,488,350	1,243,387	65.5	3,116,865	1,557,746	73.8	3,841,458
Iowa	9,757,950	6,893,383	3,333,805	466.7	3,456,619	1,673,515	125.5	2,900,820	1,250,014	88.5	3,500,616
Kansas	10,005,211	2,996,064	1,496,945	608.6	5,220,236	2,583,663	401.6	3,839,032	1,919,500	201.3	4,005,103
Kentucky	6,961,271	2,255,639	1,102,787	150.7	990,365	495,182	28.0	1,465,770	732,884	41.7	4,630,418
Louisiana	5,387,420	1,855,608	925,151	66.1	810,147	405,072	28.0	1,212,899	546,542	30.7	3,510,655
Maine	3,299,867	1,918,621	958,513	58.6	838,968	419,484	21.7	731,560	365,780	23.6	1,556,090
Maryland	3,094,808	-	-	-	1,049,363	524,646	17.5	448,610	224,305	6.3	2,345,857
Massachusetts	5,255,300	333,935	166,968	3.1	4,322,180	2,161,090	20.3	487,090	243,545	2.3	2,683,697
Michigan	11,562,296	9,222,039	4,564,957	332.3	4,938,669	2,468,431	123.7	3,133,800	1,566,275	83.9	2,962,633
Minnesota	10,344,485	8,178,850	3,844,155	532.8	2,314,658	1,157,329	127.2	1,419,220	707,097	46.4	4,635,904
Mississippi	6,635,344	-	-	-	913,040	456,820	55.9	3,389,490	1,693,940	161.1	4,484,584
Missouri	11,479,090	4,101,546	2,043,658	448.9	7,309,623	3,642,893	257.2	2,980,011	1,486,262	113.1	4,306,276
Montana	7,744,061	3,918,264	2,193,285	392.7	2,442,733	1,367,686	163.7	647,433	324,306	29.2	3,858,784
Nebraska	7,809,353	2,737,150	1,365,932	170.2	2,905,687	1,458,426	288.6	988,954	494,458	139.3	4,490,537
Nevada	4,821,864	1,571,360	1,355,536	272.2	567,128	487,083	11.5	9,600	8,300	.5	2,970,945
New Hampshire	1,843,750	857,796	422,345	24.8	192,933	96,014	2.3	-	-	-	1,325,391
New Jersey	5,054,295	2,466,551	1,233,124	34.7	2,257,052	1,051,231	24.9	19,989	9,994	-	2,759,945
New Mexico	6,030,708	3,361,190	2,065,128	273.2	1,873,692	1,139,563	122.2	603,983	367,752	28.8	2,458,265
New York	18,565,567	8,078,571	3,978,430	167.7	15,289,384	7,461,092	243.3	2,572,400	823,800	37.0	6,302,245
North Carolina	8,877,837	2,488,072	1,242,749	313.0	3,506,143	1,746,172	245.2	2,139,840	954,770	72.8	4,934,147
North Dakota	5,914,683	-	-	-	384,820	204,581	.4	-	-	-	5,710,102
Ohio	13,771,548	2,479,799	1,236,370	44.3	5,896,649	2,824,381	58.5	2,975,300	1,463,150	34.5	8,247,647
Oklahoma	8,880,547	2,671,532	1,388,451	92.1	2,178,347	1,142,458	83.4	1,407,578	729,977	43.9	5,619,661
Oregon	6,182,079	2,915,109	1,754,100	110.5	2,788,716	1,661,715	117.5	1,058,055	622,443	23.7	2,143,821
Pennsylvania	16,129,804	6,056,250	3,320,816	111.3	7,676,677	3,830,511	105.5	4,117,693	1,985,452	61.6	6,993,026
Rhode Island	1,843,750	213,888	106,944	3.4	678,336	339,168	5.6	173,727	86,864	2.1	1,310,774
South Carolina	5,103,525	261,882	106,400	27.9	4,365,561	1,749,520	269.7	983,739	372,702	92.2	2,874,903
South Dakota	6,162,747	1,384,652	777,493	188.6	127,800	80,869	36.5	772,488	423,492	57.4	4,880,893
Tennessee	7,949,380	2,260,730	1,128,235	98.8	609,776	304,883	18.3	1,421,148	710,579	53.3	5,805,683
Texas	23,506,431	12,178,430	6,072,908	685.4	7,632,063	3,772,158	453.7	5,033,235	2,484,394	319.9	11,176,971
Utah	4,274,740	2,099,593	1,471,004	138.1	1,022,986	738,448	72.1	265,120	189,950	23.3	1,875,338
Vermont	1,843,750	1,319,301	655,718	62.9	801,645	373,910	23.1	284,759	142,300	9.1	671,822
Virginia	6,887,569	2,753,546	1,373,932	109.8	2,147,636	1,073,815	108.0	1,716,452	858,226	48.7	3,581,596
Washington	5,907,615	3,882,942	2,037,427	149.9	2,020,071	1,060,771	84.7	517,744	270,660	3.8	2,538,757
West Virginia	4,107,201	863,018	431,498	42.3	734,456	367,216	17.7	721,065	360,532	17.1	2,947,955
Wisconsin	9,197,557	3,881,767	1,862,662	160.6	4,067,710	1,979,584	129.7	658,658	328,275	18.4	5,027,037
Wyoming	4,722,322	2,814,018	1,732,346	354.0	1,468,308	898,316	152.6	379,966	234,004	34.2	1,857,156
Hawaii	1,843,750	-	-	-	522,275	258,192	10.0	402,460	200,415	8.4	1,385,143
Puerto Rico	625,000	-	-	-	-	-		-	-	-	625,000
TOTALS	368,750,000	143,024,208	74,582,723	7,835.2	135,236,033	69,808,919	4,994.5	67,669,218	34,247,175	2,507.7	190,111,183

[1] APPORTIONMENTS FOR FISCAL YEARS 1936 TO 1938, INCLUSIVE.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF FEBRUARY 28, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,156,750	$ 3,156,450	105.1	$ 922,050	$ 922,050	33.2				$ 72,615
Arizona	2,569,841	2,554,087	2,205,205	163.0	627,373	330,201	32.9				34,434
Arkansas	3,352,061	2,503,236	2,485,495	260.5	800,527	798,655	87.6	$ 29,715	$ 29,499	10.9	38,412
California	7,747,928	5,819,082	5,635,900	228.4	2,062,739	2,045,845	26.9	120,920	60,500	5.2	5,683
Colorado	3,395,263	1,963,805	1,938,642	98.2	153,510	153,494	7.0				1,303,127
Connecticut	1,418,709	406,018	386,041	4.3	501,375	475,608	9.1	192,504	177,004	4.1	380,055
Delaware	900,310	417,640	410,683	45.4	432,661	353,856	21.4				135,771
Florida	2,597,144	1,518,708	1,514,871	69.2	1,017,380	1,017,380	29.8				64,893
Georgia	4,988,967	500,703	493,900	29.4	885,430	885,430	58.9	214,189	214,189	16.9	3,395,448
Idaho	2,222,747	2,221,486	2,141,178	182.9	56,927	56,927	2.7				24,642
Illinois	8,694,009	7,404,032	7,367,621	418.3	1,310,370	1,310,292	52.9				16,096
Indiana	4,941,255	2,744,027	2,515,205	115.7	2,322,341	2,322,341	112.8	37,924	37,924	6.3	65,784
Iowa	4,991,664	2,568,919	2,476,517	322.3	2,607,793	2,493,863	198.3	37,560	21,284	2.9	
Kansas	4,994,975	2,158,136	2,157,018	226.1	2,850,488	2,831,762	149.0	6,195	6,195	.5	
Kentucky	3,726,271	2,684,187	2,659,494	311.0	662,136	662,136	34.0	215,913	215,904	7.5	188,737
Louisiana	2,890,429	1,201,950	1,046,793	75.0	1,612,556	1,468,400	90.2	227,978	227,972	9.5	147,264
Maine	1,676,799	1,284,218	1,278,651	56.2	344,642	344,642	15.6	52,100	52,100	1.7	1,406
Maryland	1,750,738	378,205	376,029	17.6	564,058	564,058	10.9	468,914	369,484	12.6	441,168
Massachusetts	3,262,885	216,783	216,783	2.5	2,554,270	2,200,910	15.5	132,844	132,844	.4	712,348
Michigan	6,301,414	6,189,290	6,030,156	287.9	291,871	246,871	4.8	5,766	4,650		19,737
Minnesota	5,277,145	5,172,715	4,396,829	815.7	1,147,311	743,326	85.5	137,137	120,248	2.3	16,743
Mississippi	3,457,552	1,951,842	1,947,949	142.9	1,390,733	1,389,363	89.0	36,900	36,900	3.0	83,340
Missouri	6,012,652	3,743,122	3,707,096	707.4	2,400,423	2,192,441	69.3	38,755	36,658	1.1	76,457
Montana	3,676,416	3,351,841	3,348,936	185.1	245,028	245,028	10.2				82,451
Nebraska	3,870,739	2,235,590	2,190,400	240.1	1,314,598	1,313,095	117.6	114,105	114,105	12.2	253,139
Nevada	2,243,074	1,773,031	1,716,936	79.7	419,165	406,015	12.4	2,738	2,738	.3	117,385
New Hampshire	945,225	655,002	629,312	26.7	218,593	216,295	8.1	71,396	70,636	2.8	28,983
New Jersey	3,129,805	667,788	664,788	14.0	2,310,652	2,297,497	16.9	88,280	79,311	2.9	88,210
New Mexico	2,871,397	2,289,074	2,287,634	179.2	375,715	375,715	13.9	112,308	112,308	8.2	95,740
New York	11,046,377	9,120,833	8,698,738	133.8	2,019,260	2,019,260	34.1	295,300	295,300	2.4	33,079
North Carolina	4,720,173	2,392,128	2,359,433	171.3	2,083,786	2,083,786	109.0	235,771	199,831	10.6	77,122
North Dakota	2,867,245	1,459,542	1,456,728	215.2	1,022,734	1,019,298	90.3	156,072	156,016	20.2	235,202
Ohio	7,670,815	3,059,469	2,994,984	138.2	3,692,877	3,645,442	126.2	890,710	888,190	35.9	142,199
Oklahoma	4,580,670	2,465,491	2,440,162	240.7	1,706,367	1,698,567	144.4	256,380	251,521	17.4	190,420
Oregon	3,038,642	2,054,423	2,035,590	149.3	1,199,861	838,427	8.9	25,928	25,928	8.5	138,697
Pennsylvania	9,347,797	1,823,656	1,747,495	90.9	2,344,361	2,282,598	63.1	1,892,438	1,891,549	59.7	3,426,155
Rhode Island	989,208	1,000,291	973,602	18.8	9,520	9,520					6,086
South Carolina	2,702,012	1,237,267	1,167,394	119.0	1,084,825	1,060,044	99.9	262,823	262,603	16.7	211,970
South Dakota	2,976,454	1,945,221	1,943,858	383.0	734,747	734,747	80.5	184,020	184,020	29.8	113,830
Tennessee	4,192,460	2,200,158	2,193,276	92.0	969,291	969,291	34.6	585,902	585,902	9.8	443,991
Texas	11,989,350	11,086,439	10,131,512	1,010.2	1,963,141	1,762,009	105.6	76,061	60,475	6.0	35,355
Utah	2,067,154	1,613,492	1,475,251	163.5	431,343	399,980	26.2	76,141	76,141	3.5	115,782
Vermont	924,306	834,220	734,173	20.6	206,848	157,545	1.6	19,962	19,830	.9	12,758
Virginia	3,652,667	2,801,998	2,740,741	904.1	615,335	601,601	111.8	136,356	117,767	23.3	192,557
Washington	3,026,161	2,561,845	2,304,302	157.6	835,386	721,608	6.8				252
West Virginia	2,231,412	555,056	551,560	26.5	1,532,934	1,481,955	63.2	250,000	157,700	5.6	40,197
Wisconsin	4,823,884	4,964,889	4,404,182	328.1	509,195	368,403	15.1				51,299
Wyoming	2,219,155	1,638,931	1,622,584	123.1	552,769	533,901	17.0	54,100	27,777	12.5	34,893
District of Columbia	949,496	930,398	930,398	8.6	19,098	19,098	.2				
Hawaii	926,033	244,040	228,652	4.0	674,967	667,871	13.3				29,511
TOTALS	195,000,000	125,721,054	120,517,127	9,908.3	56,611,360	53,738,447	2,568.2	7,742,105	7,323,003	374.1	13,421,423

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF FEBRUARY 28, 1937

STATE	APPORTIONMENT	COMPLETED		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 2,372,711	$ 2,372,711	33	1	4	$ 1,457,214	$ 1,457,214	15			$ 10,140	$ 10,140			8	$ 194,552
Arizona	1,256,099	825,527	821,292	9			348,834	337,389	4								97,418
Arkansas	3,574,060	1,375,086	1,370,999	31	4		1,976,073	1,972,740	24	2	2	187,908	187,638	1		27	42,683
California	7,486,362	4,659,846	4,508,931	26	8		2,753,329	2,659,960	18								317,471
Colorado	2,631,567	1,266,630	1,239,816	21	1		938,584	938,582	8								453,170
Connecticut	1,712,684	73,479	73,479		1		779,991	758,630	5		1	76,314	76,314		1		804,261
Delaware	418,239						130,000	130,000	1								288,239
Florida	2,827,883	1,386,706	1,384,395	12	5		1,001,197	1,000,854	16	1		178,000	178,000	2			264,634
Georgia	4,895,949	30,706	30,706		2		770,657	770,657	16	2		280,154	280,154	6		30	3,814,433
Idaho	1,674,479	903,139	896,674	14	1	2	370,886	370,773	5	1		32,125	32,125			19	374,907
Illinois	10,307,184	4,012,459	4,011,582	44	1		4,896,143	4,792,943	24	5		1,429,800	1,429,800	8	1		72,859
Indiana	5,111,096	1,446,492	1,443,966	17	8		3,820,871	3,661,152	25	4	162						5,977
Iowa	5,600,679	1,984,666	1,927,117	49	6		3,417,373	3,394,668	55	3		325,615	253,980	3		8	24,914
Kansas	5,246,258	768,134	768,005	11		3	4,544,944	4,470,102	47	1		6,549	6,549			2	1,602
Kentucky	3,672,387	878,563	878,563	12	2		1,886,660	1,596,928	10	2		312,506	312,506	3	4		884,389
Louisiana	3,213,467						1,656,948	1,656,948	18	1		862,863	818,146	5	3		738,372
Maine	1,426,861	488,969	488,506	12		2	584,695	584,103	8			158,534	158,534	1	1		195,718
Maryland	2,061,751	358,388	358,388	3	3		512,104	512,104	3		17	606,404	583,936	2	2	2	607,323
Massachusetts	4,210,833	888,240	888,240	7	2		2,052,433	2,052,433	16	1		350,743	350,743	1	1		919,417
Michigan	6,765,197	3,327,414	3,278,275	34	4		3,454,277	3,454,277	11	4							32,645
Minnesota	5,395,441	2,845,392	2,775,354	59	9	37	2,509,840	2,482,579	26	3	3	58,428	58,428		1	1	79,080
Mississippi	3,241,475	712,987	712,839	20	3	1	1,881,773	1,881,773	32	1		37,270	37,270	1		4	609,593
Missouri	6,142,153	518,962	518,962	9			5,469,404	5,273,160	39	1		367,251	253,300	1		2	96,731
Montana	2,722,327	2,469,975	2,469,470	35	6		178,582	178,582	2								74,275
Nebraska	3,556,441	1,522,423	1,520,188	55	1	1	1,652,289	1,652,289	22	2	5	335,738	335,738	4		18	48,226
Nevada	887,260	690,198	690,198	7	3		211,658	193,432	1			3,630	3,630			5	
New Hampshire	822,484	341,748	341,748	3	3		235,462	235,417	3	1	1	168,326	168,326	2			76,993
New Jersey	3,983,826	619,488	619,488	4	2		2,145,687	2,134,642	15	1		570,450	570,450	2	4		659,246
New Mexico	1,725,286	1,053,490	1,053,475	13	1		673,091	670,400	6								1,410
New York	13,577,189	2,273,973	2,269,173	12	8		9,639,051	9,383,005	28	31		803,830	803,830	1	7		1,121,181
North Carolina	4,823,958	945,170	945,170	14	8		2,688,112	2,668,561	31	8	96	399,880	399,880	4	1	15	810,347
North Dakota	3,207,473	508,246	508,246	16			2,002,970	1,999,413	27	4		335,570	335,570	10			364,245
Ohio	8,439,897	327,460	261,968	2			5,247,354	5,089,353	34	5		2,265,200	2,108,142	18	2	3	980,434
Oklahoma	5,004,711	1,477,537	1,477,467	30	3		1,925,554	1,925,554	21	3		280,280	280,280	5	2	3	1,321,410
Oregon	2,334,204	878,991	878,451	8	5	2	1,491,605	1,410,005	8	1							45,748
Pennsylvania	11,483,613	1,964,345	1,760,257	26	8		7,160,942	6,739,375	44	8	9	514,840	427,628	2	2		2,556,353
Rhode Island	699,691	655,074	654,008	4	1		25,386	25,386		1		39,260	12,812		1		7,485
South Carolina	3,059,956	540,859	533,207	13	2	3	1,390,302	1,378,959	24	8		338,255	338,255	5		45	809,535
South Dakota	3,249,086	738,732	738,732	22	1	15	1,440,449	1,440,449	31	3	10	335,615	335,615	8	1	17	734,290
Tennessee	3,903,979	549,023	549,013	11	1	16	1,556,502	1,556,502	21	2	2	1,217,068	1,217,068	10		7	581,397
Texas	10,855,982	4,110,331	4,108,006	72	11		5,819,411	5,815,170	57	3		466,870	466,863			177	465,943
Utah	1,230,763	87,218	85,828	1	1		1,107,705	1,107,705	16			2,700	2,700			1	34,529
Vermont	729,857	499,385	493,786	7	5	14	226,483	186,400	3	1	1	1,380	1,300			1	48,371
Virginia	3,774,287	1,599,614	1,528,749	30	7	1	774,431	735,432	7	7		632,500	632,500	12	3	18	877,607
Washington	3,095,041	1,015,902	1,011,070	15	5	7	1,824,355	1,823,930	8	6		7,908	7,908			2	252,133
West Virginia	2,677,937						1,689,833	1,689,831	20	3		49,472	49,472	3		4	938,634
Wisconsin	5,022,683	2,028,154	1,998,580	25	4		2,935,206	2,843,954	12	3							180,149
Wyoming	1,360,841	464,745	464,638	6			648,627	648,624	6								247,579
Dist. of Columbia	410,804						425,564	396,804	3								14,000
Hawaii	453,703						522,380	453,703	5								
TOTALS	196,000,000	58,486,577	57,709,716	854	147	108	102,853,221	100,592,846	881	133	309	14,049,376	13,525,530	120	37	419	24,171,908

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Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

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CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF FEBRUARY 28, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 15,652,881	$ 8,301,392	$ 3,833,815	771.1	$ 164,711	$ 52,665	$ 112,046	5.8		$ 285,822	1.5	$ 16,076	$ 28,158
Arizona	5,211,960	2,641,935	8,984,153	5,203,675	2,605,035	543.2	24,073		24,073					8,285	12,827
Arkansas	6,748,335	3,428,049	10,964,131	6,627,424	3,367,291	619.9	136,731	80,459	55,922	6.5	$ 31,389	4,200	10.9	9,063	636
California	15,607,354	7,932,206	30,621,595	15,583,782	7,769,112	758.7	115,605		115,553					23,572	47,542
Colorado	6,874,530	3,486,006	11,255,999	6,874,530	3,459,071	633.7	6,362		6,360						20,575
Connecticut	2,865,740	1,454,868	4,514,512	2,758,269	1,312,770	74.0	59,618	59,618		.1				47,854	142,098
Delaware	1,819,088	923,395	2,783,078	1,818,804	917,472	130.0								284	5,923
Florida	5,231,834	2,661,343	8,924,696	5,175,534	2,358,763	306.4	318,779		294,464	3.5	56,300		.3		8,116
Georgia	10,091,185	5,113,491	13,294,342	9,269,813	3,256,698	754.2	1,143,612	408,690	734,922	13.4	131,952	143,474	7.8	280,731	978,396
Idaho	4,486,249	2,277,486	7,108,644	4,477,339	2,185,517	501.4	54,194		52,034	.1		1,886		8,910	38,049
Illinois	17,570,770	8,921,401	26,364,669	17,311,651	7,968,903	715.6	1,234,717	222,449	885,007	13.2	17,560	7,100	7.5	19,110	60,391
Indiana	10,037,843	5,088,963	15,546,336	9,895,839	4,837,453	482.7	227,844	129,083	98,761	2.4		58,728		12,921	94,021
Iowa	10,055,660	5,118,361	15,586,846	10,055,660	4,921,761	1,222.3	209,734		196,600	5.0					
Kansas	10,089,604	5,117,675	15,427,787	10,089,600	5,010,191	1,132.3	115,038		78,627	13.1		20,885	..	3	7,972
Kentucky	7,517,359	3,818,311	12,173,096	7,504,603	3,711,004	812.7	82,002		82,002	2.8	10,217	15,467		2,539	9,838
Louisiana	5,828,591	2,963,932	9,147,543	5,763,651	2,715,610	259.5	215,480		215,459	11.5				64,940	32,863
Maine	3,369,917	1,711,586	5,248,447	3,348,497	1,672,204	193.5	43,100	15,000	26,540	1.4				8,420	12,841
Maryland	3,564,527	1,810,058	5,659,409	3,475,617	1,103,886	152.2	68,343		68,343	1.5		285,453	7.3	88,910	352,377
Massachusetts	6,597,100	3,350,474	10,399,577	6,552,734	3,094,340	115.5						91,944	.1	44,367	164,190
Michigan	12,736,227	6,452,568	20,694,553	12,736,227	6,306,320	768.1	96,788		96,788			40,000	.4		9,460
Minnesota	10,656,569	5,425,551	16,211,829	10,514,422	4,850,678	1,641.9	372,262	58,110	268,494	7.2		34,800	.6	84,037	271,578
Mississippi	6,978,675	3,540,227	12,753,162	6,784,988	2,991,841	723.8	651,325	142,960	491,245	21.4	3,960	20,523	.7	46,767	36,618
Missouri	12,180,306	6,173,740	18,200,704	12,105,337	4,989,769	1,441.2	1,130,984		1,127,050	6.4				74,969	56,920
Montana	7,439,748	3,769,734	11,764,936	7,425,476	3,649,311	1,058.4	56,546		56,546					14,272	63,877
Nebraska	7,828,961	3,964,364	12,841,101	7,812,918	3,617,217	1,018.4	251,824		251,824	27.5		68,900	5.9	16,043	26,423
Nevada	4,545,917	2,302,356	7,065,551	4,545,917	2,268,591	758.8	25,004		25,004			1,650			7,111
New Hampshire	1,909,839	969,462	2,999,744	1,904,951	949,520	78.3	4,169		4,169					4,888	15,773
New Jersey	6,346,039	3,220,879	9,096,652	6,045,652	2,452,166	85.9	1,218,088	120,518	663,430	10.3	88,486	9,212	.6	91,384	96,071
New Mexico	5,792,935	2,941,700	8,784,010	5,731,884	2,813,448	743.9	107,130		107,130	5.9	17,267	2,709		43,784	18,412
New York	22,330,101	11,327,921	39,434,063	21,782,046	10,475,519	821.3	1,233,880	518,810	631,049	3.9	23,246	169,100	.1	5,999	52,253
North Carolina	9,522,293	4,840,941	14,965,034	9,210,800	4,527,830	1,345.8	553,051	291,910	261,141	21.5				19,582	51,971
North Dakota	5,804,448	2,938,967	8,588,055	5,586,084	2,228,278	2,126.9	226,881	106,183	120,698	16.3	15,703	182,906	28.0	96,478	407,085
Ohio	15,484,592	7,865,012	24,755,327	15,379,719	7,519,924	792.1	275,374	40,500	205,957	4.0	58,056	87,583	4.2	6,318	51,548
Oklahoma	9,216,798	4,685,180	14,647,853	9,214,657	4,330,553	805.5	272,992		272,992	1.5				2,141	81,635
Oregon	6,106,896	3,097,814	9,946,206	6,048,081	2,935,541	468.0	64,223		64,223	.8	55,000	26,981	.7	3,815	71,070
Pennsylvania	18,891,004	9,590,788	28,963,929	18,536,691	8,906,792	1,054.1	395,207	53,214	321,211	3.4	168,454	82,737	3.7	132,644	280,048
Rhode Island	1,998,708	1,014,572	3,150,270	1,998,708	1,012,094	89.1									2,478
South Carolina	5,459,165	2,770,954	7,985,664	5,211,885	2,453,069	618.5	359,440	161,980	197,260	9.4	42,400	56,356	1.7	42,900	64,269
South Dakota	6,011,479	3,047,643	9,266,230	5,834,013	2,855,879	1,571.6	228,517	77,600	150,917	29.5	5,400	8,671	6.1	94,466	32,176
Tennessee	8,492,619	4,302,991	13,667,901	8,490,706	4,135,256	492.7	140,180		113,164	3.6		46,727	5.9	1,913	7,845
Texas	24,244,024	12,291,253	38,002,703	24,189,662	11,892,458	2,780.0	124,314	14,797	109,197	2.0		188,895	8.0	39,565	100,703
Utah	4,194,708	2,132,691	7,408,593	4,193,208	2,132,691	590.9								1,500	
Vermont	1,867,573	948,007	3,166,359	1,867,452	940,847	141.0								121	7,160
Virginia	7,416,757	3,765,387	11,597,217	7,294,582	3,382,605	614.5	292,292	82,461	204,199	30.5	9,045	85,002	5.2	30,668	93,582
Washington	6,115,867	3,106,412	9,399,029	6,112,042	3,026,509	302.7	46,596		46,596			14,823	.3	3,825	18,484
West Virginia	4,474,234	2,280,335	6,472,133	4,342,391	1,791,107	212.9	375,487	54,892	297,549	9.4		66,425	.7	76,951	125,254
Wisconsin	9,724,881	4,941,837	15,436,536	9,720,705	4,884,968	619.7	12,330		12,330		4,176	2,720			41,819
Wyoming	4,501,327	2,287,712	6,889,086	4,451,922	2,228,182	1,037.7	5,781	5,780		.3	11,618	21,300	2.5	31,806	38,230
District of Columbia	1,918,469	973,842	2,887,584	1,918,469	968,979	22.3						4,863			
Hawaii	1,871,062	949,778	2,680,982	1,857,812	264,953	51.1	650,838		646,859	4.7		14,000		13,250	23,966
TOTALS	394,000,000	200,000,000	629,380,737	388,935,821	183,883,791	35,056.0	13,391,446	2,697,679	9,793,735	359.8	750,429	2,151,842	110.8	1,616,071	4,170,632

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 2 APRIL 1937

BINS USED IN PROPORTIONING AGGREGATE FOR CONCRETE TEST SECTIONS

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 2 April 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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THE EFFECT OF VIBRATION ON THE STRENGTH AND UNIFORMITY OF PAVEMENT CONCRETE

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS [1]

Reported by F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer

IN THIS INVESTIGATION the effects of surface vibration on pavement concrete were measured by comparing the strength and other properties of vibrated concrete with similar properties of a standard-finished base mix containing 6 sacks of cement per cubic yard of concrete and sufficient water to produce a slump of 2½ inches. Two types of coarse aggregates—a typical river gravel and a typical limestone—and several forms of surface vibrating equipment were used, including vibrating screeds and vibrating pans resting directly on the concrete.

In the vibrated concrete the proportions of the base mix were varied for the purpose of determining: (1) The saving in cost that might be possible through a reduction in cement content while maintaining the water-cement ratio constant; and (2) the improvement in quality that could be effected by reducing the water-cement ratio while maintaining the cement content constant.

The possibility of using high frequency vibration as an aid in placing pavement concrete was first investigated by the Bureau of Public Roads in 1931. The equipment used at that time consisted of a standard, double-screed finishing machine, with two electric vibrators mounted on the front screed and one on the rear screed. An experimental concrete pavement, 9 feet wide and approximately 1,000 feet long, was constructed at Arlington, Va. This pavement consisted of a series of sections, each 9 feet long, with means provided for obtaining large-size test slabs from each section. It was felt that the true effect of this method of placing and finishing concrete pavements could only be determined by the construction of full-size sections, using regulation contractors' equipment. The results of these studies, including a full description of the procedure, have been published.[2]

CONCLUSIONS DRAWN FROM TEST RESULTS

The data from these tests indicate that:

1. For a given water-cement ratio a saving of approximately 10 percent in the amount of cement can be effected by the use of vibration without sacrificing strength and uniformity.

2. For a given cement content an increase in flexural and compressive strength of approximately 10 percent can be obtained by the use of vibration.

3. With the methods of surface vibration investigated the best results will be obtained with concrete having a consistency corresponding to a slump of approximately 1 inch.

4. The uniformity of concrete having a slump of approximately 1 inch (that is, freedom from segregation, honeycombed areas, etc.) is markedly improved by surface vibration.

5. The effect of surface vibration on strength and uniformity seems to be about the same for concrete containing gravel as coarse aggregate as for concrete containing crushed stone.

6. The vibrating screed and the vibrating pan mounted independently of the screeds seem to be about equally effective in producing the results enumerated in the above conclusions.

These tests indicated that vibration caused sufficient improvement in quality to warrant further studies in which improved types of vibratory equipment could be utilized. The present report discusses the results of these further tests, draws certain conclusions, and makes suggestions regarding the revision of current specifications so as to make it possible to utilize to the best advantage the equipment now available. The data also reveal some of the deficiencies in the present methods of distributing concrete to the subgrade. Entirely apart from the question of vibration, the results indicate the great desirability of improvement in this regard.

Results of tests on 270 sections, each 8 feet long and 10 feet wide, are presented and discussed in this report. These test sections formed the equivalent of a continuous pavement nearly ½ mile long. In most cases the thickness was the same as that previously used (7 inches). However, slabs 10 inches thick were cast in a few instances for purposes of comparison. The width was changed from the 9 feet previously used to 10 feet in order to permit taking 5 slabs each 24 inches wide from each test section instead of 4 slabs each 27 inches wide. Two coarse aggregates, a typical river gravel and a typical crushed stone, were used. Four different types of vibrating equipment, including all of the types commercially available at the time, were investigated, using paving mixtures varying considerably in harshness and consistency. The construction, curing, and test procedures followed, in general, those previously employed and will not be described in detail, except where necessary to describe a change from previous practice.

FOUR TYPES OF VIBRATING EQUIPMENT STUDIED

A single lot of portland cement meeting the usual requirements was used throughout the work. The cement came from one bin and was all shipped at one time. The shipment was made under the supervision of an inspector representing the National Bureau of Standards. The fine aggregate was Potomac River sand having a fineness modulus of approximately 3.0.

[1] Acknowledgment is made to the following companies, which loaned equipment used in these tests:
Jaeger Machine Co., Columbus, Ohio.
Blaw-Knox Co., Pittsburgh, Pa.
Baily Vibrator Co., Philadelphia, Pa.

[2] The Effect of Vibration and Delayed Finishing on Pavement Slabs, by F. H. Jackson and W. F. Kellermann. PUBLIC ROADS, vol. 14, no. 8, October 1933.

133007—37——1

FORMS IN PLACE AND SUBGRADE PREPARED, READY FOR CONCRETE TO BE DEPOSITED FOR A 10-INCH TEST SECTION.

APPEARANCE OF A 7-INCH TEST SECTION AFTER FIRST BATCH OF CONCRETE HAS BEEN STRUCK OFF AND AFTER INSTALLATION OF WOODEN SEPARATORS.

The coarse aggregates were Potomac River gravel and a crushed stone from Riverton, Virginia. They were used in approximately equal amounts and were handled in two separated sizes, 2-inch to 1-inch, and 1-inch to No. 4. The two sizes were combined in each batch in the proportion of 60 percent of the larger to 40 percent of the smaller size. The grading and physical characteristics of the aggregates are shown in table 1.

The major portion of the work was confined to a study of two well-known mechanical finishing machines, equipped in each case to operate either with or without vibration. Machine A, used on 144 sections, was of the type studied in the 1931 tests, except that a wider and heavier front screed was used. The type of finisher represented by machine B, used on 106 sections, had not been equipped with a vibrating mechanism at the time the earlier tests were run. Two other devices submitted for demonstration purposes were used on the remaining 20 sections. A brief description of each machine follows.

TABLE 1.—*Properties of aggregates used in experimental concrete sections*

	Fine aggregate	Coarse aggregate	
		Gravel	Crushed stone
PHYSICAL PROPERTIES			
Bulk specific gravity	2.56	2.57	2.76
Absorption, percent	1.40	1.14	.32
Weight per cubic foot (dry-rodded), pounds	110	109	103
Voids, percent	31	32	40
Wear (Deval), do		8.4	3.4
SIEVE ANALYSES			
Retained on—			
1½-inch sieve, percent		25	18
¾-inch sieve, do		64	71
⅜-inch sieve, do		87	92
No. 4 sieve, do	5	98	99
No. 8 sieve, do	22	100	100
No. 14 sieve, do	34	100	100
No. 28 sieve, do	51	100	100
No. 48 sieve, do	84	100	100
No. 100 sieve, do	97	100	100
Fineness modulus	2.93	7.74	7.80

Machine A was a mechanical finishing machine equipped with two tilting screeds. Power for operating the screeds and for driving the machine was furnished by a gasoline engine mounted on the finisher. The regular front screed was replaced by an 18-inch bull-nose screed. Two electric vibrators were mounted on the front screed and one on the rear screed. They were driven at a speed of 4,000 r. p. m. by a portable gas-electric generator set mounted on the finisher. The normal forward speed of the machine was 8 feet per minute.

Except for the fact that the screeds were vibrated, this machine was operated in most cases in the usual way, that is, two complete passes over the concrete.

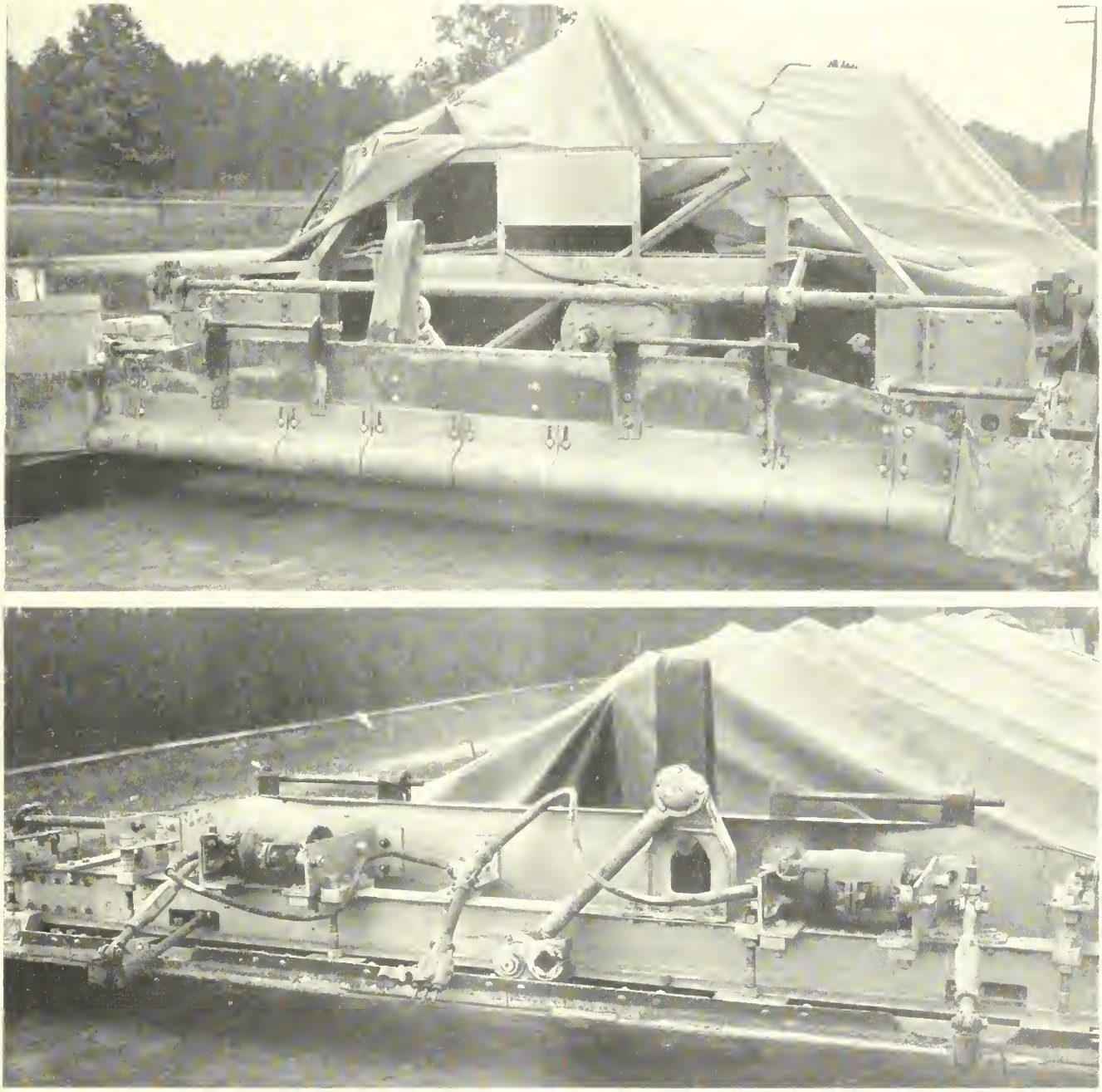
Machine B was an all-electric mechanical finishing machine equipped with two screeds and a vibrating pan mounted between them. The vibrating pan was shaped like a trough and made contact with the concrete for practically the full width of the pavement. It was adjustable so that it could be raised clear of the concrete or could be lowered into the concrete a predetermined depth. It was equipped with two electric vibrators of adjustable amplitude operating at 3,600 r. p. m. There were no vibrators on the screeds. Power for running the machine, operating the screeds, and running the vibrators was supplied by a gas-electric unit mounted on the finisher.

The concrete was struck off approximately one-fourth inch high by the front screed and compacted with the vibrator. The vibrating pan was submerged in the concrete to the extent that its lower edge was about one-fourth inch below the elevation of the top of the side forms. In most cases the vibrator was operated during the backward pass of the finisher as well as during the forward pass. By running the vibrator during the backward pass, a ridge of mortar was left at the beginning of the section which materially aided in the finishing operations. The height above the forms to which the concrete was struck varied with the consistency and harshness of the mix. The objective sought was to have sufficient surplus concrete so that, after vibrating, the surface would be at grade. For the second and subsequent passes the machine was operated in the usual way without vibration.

Machine C was not a regular finishing machine but rather a single, hand-operated screed with two vibrators attached. It consisted of a 4-inch wooden plank 15 inches wide, shod with sheet steel, and approximately 12 feet long. Handles were provided at both ends for manipulation. This device was vibrated at a frequency of 3,800 r. p. m. by means of a small gasoline engine attached to the screed.

The concrete was leveled off about 2 inches high by shovels and a pass made with the vibratory screed, the screed being pushed ahead by finishing machine A. The vibrating screed was moved ahead and the pavement finished by machine A except that the bull-nose front screed was replaced by the regular 12-inch screed. The 12-inch screed weighed approximately 450 pounds less than the bull-nose screed with vibrators.

Machine D consisted of a two-wheel steel carriage, to which was attached a steel trough 12 inches wide, mounted at right angles to the axis of the pavement. The trough could be lowered into the concrete a pre-



BULL-NOSE FRONT SCREED ON FINISHING MACHINE A. UPPER, FRONT VIEW; LOWER, REAR VIEW.

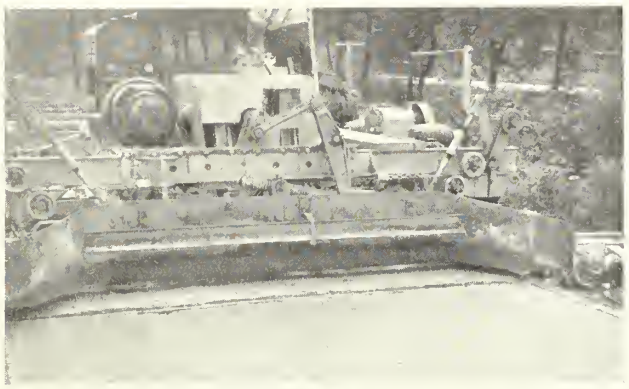
determined depth in a manner similar to the operation of the vibrator on machine B. Vibrations were imparted to the trough by means of a 3¼-inch steel tube revolving at a speed of approximately 3,800 r.p.m. The tube was mounted eccentrically on a shaft which was attached directly to the trough by suitable bearings. The eccentricity of the tube could be varied, resulting in varying amplitude of vibration. Power was supplied directly to the shaft from a gasoline engine mounted on the carriage through a V-belt drive. The machine was attached to the rear of finishing machine A.

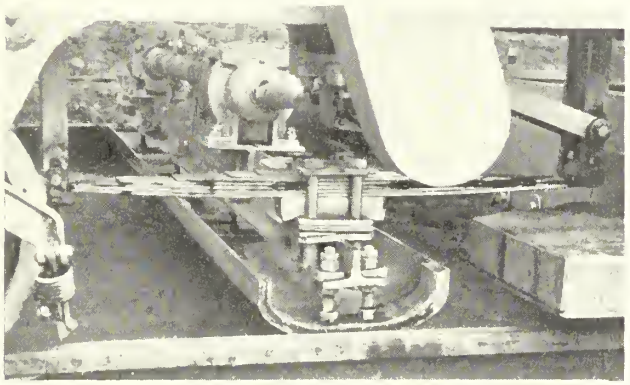
The concrete was struck off approximately three-fourths inch high by machine A operating with a flat front screed. This was followed by one pass with the vibrating pan, the general procedure being the same as with machine B except that in no case was the vibrator operated on the backward pass. The final finish was obtained by making one or two passes with finishing machine A without vibration.

Table 2 shows the order in which the vibrators were used as well as the procedure followed for each group of sections. It will be observed that only machine A was used in series A. Machines A and B were used in series B, and machines C and D were used in series C.

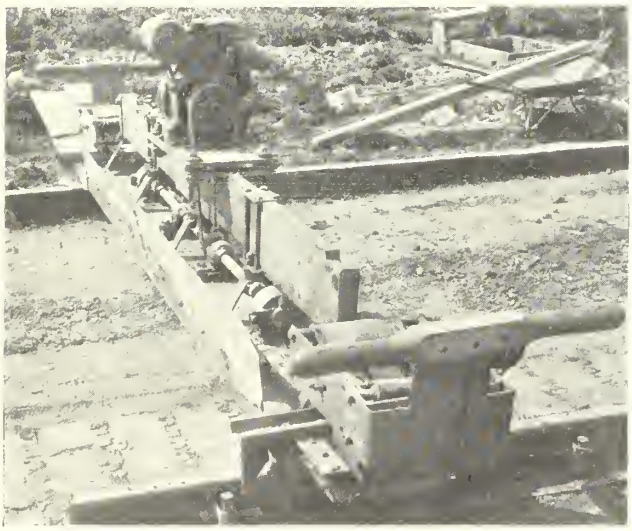
SEVERAL DIFFERENT CONCRETE MIXES USED IN SLABS

As in the previous tests, the effect of vibration was measured by comparing the strength and other properties of the vibrated concrete with similar properties of a standard-finished paving mix having a cement content of 6 sacks per cubic yard, and containing suffi-





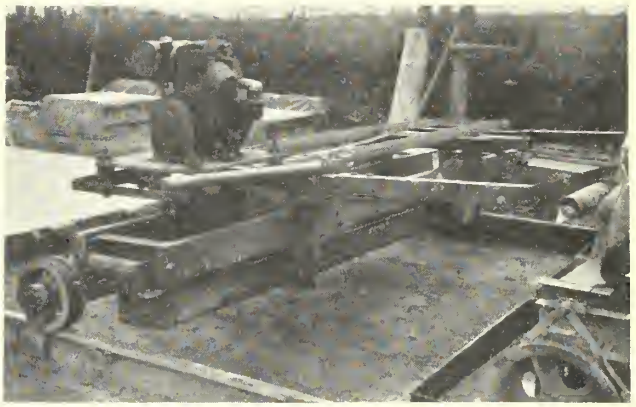
FINISHING MACHINE B. UPPER, FRONT VIEW; LOWER, SIDE VIEW, SHOWING MOUNTING OF VIBRATING PAN.



FINISHING MACHINE C—A HAND-OPERATED SCREED VIBRATED BY MEANS OF THE SMALL GASOLINE ENGINE SHOWN.

cient water to give a slump of about 2½ inches. In the following discussion this will be referred to as the standard or "base" mix.

For the first 42 sections, constituting series A, the proportions of the base mix were designed to give a workability factor (b/b_o) of approximately 0.76 (dry and rodded) for both gravel and stone. Corresponding values for the sand-cement ratios (a/c) by absolute volume were 2.10 for gravel and 2.53 for stone. This mix proved somewhat harsh for standard finishing, necessitating a slight increase in the amount of sand. The revised proportions for the base mix as used in



FINISHING MACHINE D, SHOWING THE PAN VIBRATOR.

series B and C called for a workability factor of 0.73, with sand-cement ratios of 2.24 for gravel and 2.67 for stone. (Explanations of symbols b/b_o, a/c, etc., are given in the footnote to table 3.)

TABLE 2.—*Type and operation of vibrating equipment*

SERIES A—MACHINE A

Section no.	Figures in which data are shown	Coarse aggregate	Operation of finishing machine
1–6	12	Gravel	Both screeds vibrating; 2 passes.
7–12		do	Front screed only vibrating; 2 passes.
13–18		do	Do.
19–24	12	do	Both screeds vibrating; 2 passes.
25–30	12	do	Do.
31–36	12	Stone	Do.
37–42	12	do	Do.

SERIES B—MACHINE A

43–49	6–11, inclusive	Stone	Both screeds vibrating; 2 passes.
50–56	do	do	Do.
57–63	do	Gravel	Do.
64–70	do	do	Do.
177–182	6–13, inclusive	Stone	Do.
183–189	6–11, inclusive	Gravel	Do.
190–196	do	do	Do.
197–202	6–12, inclusive	do	Do.
203–208 [1]	6–9, inclusive	do	Do.
209–214	6–12, inclusive	do	Do.
215–220		do	Front screed only vibrating; 2 passes.
221–226	6–11, inclusive	do	Both screeds vibrating; 2 passes.
227–232	6–13, inclusive	Stone	Do.
233–238		do	Front screed only vibrating; 2 passes.
239–244	6–11, inclusive	do	Both screeds vibrating; 2 passes.
245–250 [1]	6–9, inclusive	do	Do.

SERIES B—MACHINE B

71–76		Gravel	Pan vibrating forward only; 1 pass.
77–82		do	Do.
83–88		do	Do.
89–94		Stone	Do.
95–100 [1]	6–9, inclusive	do	Pan vibrating forward and back.
101–106 [1]	do	Gravel	Do.
107–112	6–11, inclusive	do	Do.
113–119	do	do	Do.
120–126	do	do	Do.
127–132	do	do	Do.
133–138	do	do	Do.
139–144	6–11, inclusive, 13	Stone	Do.
145–150	do	do	Do.
151–156		do	Pan vibrating forward only; 1 pass.
157–162	6–11, inclusive	do	Pan vibrating forward and back.
163–169	do	do	Do.
170–176	do	do	Do.

SERIES C—MACHINES C AND D

251–254	13	Stone	Machine C, 1 pass.
255–258	13	do	Do.
259–264	13	do	Machine D, 1 pass.
265–270	13	do	Do.

[1] Depth of section was 10 inches.

TABLE 3.—*Data on mixes* [1]

Section no.	Date laid (1934)	Proportions by weight	Coarse aggregate	Slump	W/C	a/c	b/b₀	M/V	Cement factor
		Pounds		*Inches*					*Sacks per cubic yard*
1-S	June 26	94:165:371	Gravel	2.9	0.68	2.10	0.75	2.02	6.0
2-S	do	94:165:371	do	1.6	.62	2.10	.76	1.96	6.1
3-V	do	94:165:371	do	1.6	.61	2.10	.76	1.95	6.1
4-V	do	94:165:425	do	2.2	.68	2.10	.80	1.76	5.6
5-V	do	94:165:452	do	1.8	.68	2.10	.83	1.66	5.4
6-V	do	94:165:480	do	1.5	.68	2.10	.85	1.56	5.2
7-S	June 27	94:165:371	do	3.0	.68	2.10	.75	2.02	6.0
8-S	do	94:165:371	do	1.3	.59	2.10	.77	1.94	6.1
9-V	do	94:165:371	do	1.2	.59	2.10	.77	1.94	6.1
10-V	do	94:165:425	do	2.5	.68	2.10	.80	1.76	5.6
11-V	do	94:165:452	do	2.1	.68	2.10	.83	1.66	5.4
12-V	do	94:165:480	do	1.8	.68	2.10	.85	1.56	5.2
13-S	July 5	94:165:371	do	2.4	.70	2.10	.75	2.04	5.9
14-S	do	94:165:371	do	1.1	.63	2.10	.76	1.97	6.0
15-V	do	94:165:371	do	1.0	.63	2.10	.76	1.97	6.0
16-V	do	94:165:425	do	1.5	.70	2.10	.80	1.78	5.5
17-V	do	94:165:452	do	1.2	.70	2.10	.82	1.67	5.3
18-V	do	94:165:480	do	.8	.70	2.10	.84	1.57	5.2
19-S	July 6	94:165:371	do	2.5	.71	2.10	.75	2.05	5.9
20-S	do	94:165:371	do	1.4	.65	2.10	.76	1.99	6.0
21-V	do	94:165:371	do	1.4	.65	2.10	.76	1.99	6.0
22-V	do	94:165:425	do	1.8	.71	2.10	.80	1.78	5.5
23-V	do	94:165:452	do	1.2	.71	2.10	.82	1.68	5.3
24-V	do	94:165:480	do	.8	.71	2.10	.84	1.58	5.2
25-S	July 9	94:165:371	do	2.2	.68	2.10	.75	2.02	6.0
26-S	do	94:165:371	do	1.2	.62	2.10	.76	1.96	6.1
27-V	do	94:165:371	do	1.4	.62	2.10	.76	1.96	6.1
28-V	do	94:165:425	do	1.5	.68	2.10	.80	1.76	5.6
29-V	do	94:165:452	do	1.2	.68	2.10	.83	1.65	5.4
30-V	do	94:165:480	do	.9	.68	2.10	.85	1.56	5.2
31-S	July 11	94:198:350	Stone	2.7	.73	2.53	.76	1.81	6.0
32-S	do	94:198:350	do	1.4	.69	2.53	.76	1.78	6.1
33-V	do	94:198:350	do	1.4	.69	2.53	.76	1.78	6.1
34-V	do	94:198:402	do	2.1	.74	2.53	.81	1.58	5.6
35-V	do	94:198:428	do	1.4	.74	2.53	.84	1.49	5.4
36-V	do	94:198:453	do	1.2	.74	2.53	.86	1.40	5.3
37-S	July 12	94:198:350	do	2.4	.73	2.53	.76	1.81	6.0
38-S	do	94:198:350	do	1.4	.69	2.53	.76	1.78	6.1
39-V	do	94:198:350	do	1.6	.69	2.53	.76	1.78	6.1
40-V	do	94:198:402	do	1.4	.73	2.53	.81	1.58	5.6
41-V	do	94:198:428	do	1.1	.73	2.53	.84	1.48	5.5
42-V	do	94:198:453	do	.8	.73	2.53	.86	1.40	5.3
43-S	July 19	94:209:340	do	2.4	.77	2.67	.73	1.95	5.9
44-S	do	94:209:340	do	1.0	.71	2.67	.73	1.90	6.0
45-V	do	94:209:340	do	1.0	.71	2.67	.73	1.90	6.0
46-V	do	94:194:360	do	1.4	.73	2.47	.77	1.74	6.0
47-V	do	94:194:360	do	1.0	.70	2.47	.78	1.71	6.0
48-V	do	94:177:378	do	1.2	.73	2.27	.81	1.59	6.0
49-V	do	94:177:378	do	.8	.70	2.27	.82	1.57	6.0
50-S	July 20	94:209:340	do	2.4	.77	2.67	.73	1.95	5.9
51-S	do	94:209:340	do	1.6	.73	2.67	.73	1.92	6.0
52-V	do	94:209:340	do	1.2	.73	2.67	.73	1.92	6.0
53-V	do	94:194:360	do	1.4	.72	2.47	.77	1.73	6.0
54-V	do	94:194:360	do	.8	.68	2.47	.78	1.70	6.0
55-V	do	94:177:378	do	1.6	.73	2.27	.81	1.59	6.0
56-V	do	94:177:378	do	.9	.69	2.27	.82	1.56	6.0
57-S	July 23	94:176:354	Gravel	2.6	.74	2.24	.72	2.24	5.9
58-S	do	94:176:354	do	1.2	.66	2.24	.73	2.16	6.1
59-V	do	94:176:354	do	1.2	.66	2.24	.73	2.16	6.1
60-V	do	94:163:376	do	1.4	.67	2.08	.76	1.98	6.0
61-V	do	94:163:376	do	.9	.64	2.08	.77	1.95	6.0
62-V	do	94:146:392	do	2.0	.67	1.88	.79	1.81	6.0
63-V	do	94:146:392	do	.8	.62	1.88	.80	1.77	6.0
64-S	July 24	94:176:354	do	2.8	.75	2.24	.71	2.25	5.9
65-S	do	94:176:354	do	1.4	.66	2.24	.73	2.16	6.1
66-V	do	94:176:354	do	1.2	.66	2.24	.73	2.16	6.1
67-V	do	94:163:376	do	1.6	.67	2.08	.76	1.98	6.0
68-V	do	94:163:376	do	.9	.64	2.08	.77	1.95	6.0
69-V	do	94:146:392	do	1.5	.65	1.88	.80	1.79	6.0
70-V	do	94:146:392	do	.9	.61	1.88	.81	1.76	6.0
71-S	July 26	94:176:354	do	2.6	.73	2.24	.72	2.23	6.0
72-S	do	94:176:354	do	1.1	.64	2.24	.73	2.14	6.1
73-V	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1
74-V	do	94:176:409	do	1.7	.73	2.24	.77	1.93	5.5
75-V	do	94:176:436	do	1.1	.73	2.24	.79	1.81	5.4
76-V	do	94:176:463	do	.9	.73	2.24	.82	1.71	5.2
77-S	July 27	94:176:354	do	2.5	.73	2.24	.72	2.23	6.0
78-S	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1
79-V	do	94:176:354	do	1.0	.64	2.24	.73	2.14	6.1
80-V	do	94:176:409	do	2.4	.73	2.24	.77	1.93	5.5
81-V	do	94:176:436	do	1.7	.73	2.24	.79	1.81	5.4
82-V	do	94:176:463	do	1.1	.73	2.24	.82	1.71	5.2
83-S	July 30	94:176:354	do	3.0	.71	2.24	.72	2.21	6.0
84-S	do	94:176:354	do	1.4	.61	2.24	.74	2.12	6.1
85-V	do	94:176:354	do	1.4	.61	2.24	.74	2.12	6.1
86-V	do	94:192:409	do	1.5	.71	2.45	.76	2.00	5.4
87-V	do	94:200:436	do	1.2	.71	2.55	.77	1.91	5.2
88-V	do	94:209:463	do	.9	.71	2.67	.79	1.85	5.0
89-S	July 31	94:209:340	Stone	2.4	.79	2.67	.72	1.96	5.9
90-S	do	94:209:340	do	1.4	.73	2.67	.73	1.92	6.0
91-V	July 31	94:209:340	Stone	1.2	0.73	2.67	0.73	1.92	6.0
92-V	do	94:209:391	do	1.6	.79	2.67	.78	1.70	5.6
93-V	do	94:209:417	do	1.1	.79	2.67	.81	1.60	5.4
94-V	do	94:209:443	do	.7	.79	2.67	.83	1.51	5.2
95-S	Aug. 1	94:209:340	do	3.0	.84	2.67	.71	2.00	5.8
96-S	do	94:209:340	do	1.3	.74	2.67	.73	1.92	6.0
97-V	do	94:209:340	do	1.2	.74	2.67	.73	1.92	6.0
98-V	do	94:209:391	do	2.1	.84	2.67	.77	1.74	5.5
99-V	do	94:209:417	do	2.0	.84	2.67	.80	1.63	5.3
100-V	do	94:209:443	do	1.1	.84	2.67	.82	1.53	5.2
101-S	Aug. 2	94:176:354	Gravel	2.4	.72	2.24	.72	2.22	6.0
102-S	do	94:176:354	do	1.0	.63	2.24	.73	2.13	6.1
103-V	do	94:176:354	do	1.0	.63	2.24	.73	2.13	6.1
104-V	do	94:176:409	do	1.9	.72	2.24	.77	1.93	5.5
105-V	do	94:176:436	do	1.5	.72	2.24	.80	1.80	5.4
106-V	do	94:176:463	do	1.1	.72	2.24	.82	1.70	5.2
107-S	Aug. 3	94:176:354	do	2.6	.70	2.24	.72	2.20	6.0
108-S	do	94:176:354	do	1.2	.61	2.24	.74	2.12	6.1
109-V	do	94:176:354	do	1.0	.61	2.24	.74	2.12	6.1
110-V	do	94:176:409	do	1.8	.70	2.24	.77	1.91	5.6
111-V	do	94:176:436	do	1.4	.70	2.24	.80	1.79	5.4
112-V	do	94:176:463	do	1.0	.70	2.24	.82	1.68	5.2
113-S	Aug. 6	94:176:354	do	2.2	.73	2.24	.72	2.23	6.0
114-S	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1
115-V	do	94:176:354	do	1.0	.64	2.24	.73	2.14	6.1
116-V	do	94:163:376	do	1.4	.67	2.08	.76	1.98	6.0
117-V	do	94:163:376	do	.9	.63	2.08	.77	1.95	6.0
118-V	do	94:146:392	do	1.2	.64	1.88	.80	1.78	6.0
119-V	do	94:146:392	do	.6	.64	1.88	.80	1.78	6.0
120-S	Aug. 7	94:176:354	do	3.0	.76	2.24	.71	2.26	5.9
121-S	do	94:176:354	do	1.2	.66	2.24	.73	2.16	6.1
122-V	do	94:176:354	do	1.0	.66	2.24	.73	2.16	6.1
123-V	do	94:163:376	do	1.6	.67	2.08	.76	1.98	6.0
124-V	do	94:163:376	do	.8	.63	2.08	.77	1.95	6.0
125-V	do	94:146:392	do	1.4	.66	1.88	.80	1.80	6.0
126-V	do	94:146:392	do	.8	.64	1.88	.80	1.78	6.0
127-S	Aug. 8	94:176:354	do	2.8	.75	2.24	.71	2.26	5.9
128-S	do	94:176:354	do	1.2	.68	2.24	.73	2.18	6.0
129-V	do	94:176:354	do	1.2	.68	2.24	.73	2.18	6.0
130-V	do	94:176:409	do	2.2	.75	2.24	.77	1.95	5.5
131-V	do	94:176:436	do	1.9	.75	2.24	.79	1.83	5.3
132-V	do	94:176:463	do	1.3	.75	2.24	.81	1.72	5.2
133-S	Aug. 9	94:176:354	do	2.4	.75	2.24	.71	2.25	5.9
134-S	do	94:176:354	do	1.2	.69	2.24	.72	2.19	6.0
135-V	do	94:176:354	do	1.2	.69	2.24	.72	2.19	6.0
136-V	do	94:192:409	do	1.1	.75	2.45	.75	2.03	5.4
137-V	do	94:200:436	do	.8	.75	2.55	.77	1.95	5.2
138-V	do	94:209:463	do	.8	.78	2.67	.78	1.90	4.9
139-S	Aug. 10	94:209:340	Stone	3.2	.82	2.67	.72	1.98	5.9
140-S	do	94:209:340	do	1.2	.71	2.67	.73	1.90	6.0
141-V	do	94:209:340	do	1.2	.71	2.67	.73	1.90	6.0
142-V	do	94:209:391	do	2.4	.82	2.67	.78	1.72	5.5
143-V	do	94:209:417	do	1.5	.82	2.67	.80	1.62	5.4
144-V	do	94:209:443	do	1.0	.82	2.67	.83	1.52	5.2
145-S	Aug. 13	94:209:340	do	2.6	.80	2.67	.72	1.97	5.9
146-S	do	94:209:340	do	1.2	.70	2.67	.74	1.89	6.0
147-V	do	94:209:340	do	1.0	.70	2.67	.74	1.89	6.0
148-V	do	94:209:391	do	2.0	.80	2.67	.78	1.71	5.5
149-V	do	94:209:417	do	1.4	.80	2.67	.81	1.60	5.4
150-V	do	94:209:443	do	1.0	.80	2.67	.83	1.51	5.2
151-S	Aug. 14	94:209:340	do	2.7	.80	2.67	.72	1.97	5.9
152-S	do	94:209:340	do	1.1	.70	2.67	.74	1.89	6.0
153-V	do	94:209:340	do	1.2	.70	2.67	.74	1.89	6.0
154-V	do	94:224:391	do	1.8	.80	2.86	.77	1.77	5.4
155-V	do	94:231:417	do	1.2	.80	2.94	.79	1.69	5.2
156-V	do	94:239:443	do	.8	.80	3.04	.80	1.62	5.0
157-S	Aug. 15	94:209:340	do	2.8	.81	2.67	.72	1.98	5.9
158-S	do	94:209:340	do	1.1	.70	2.67	.74	1.89	6.0
159-V	do	94:209:340	do	1.0	.70	2.67	.74	1.89	6.0
160-V	do	94:224:391	do	1.8	.81	2.86	.76	1.78	5.4
161-V	do	94:231:417	do	1.3	.81	2.94	.78	1.69	5.2
162-V	do	94:239:443	do	.8	.81	3.04	.80	1.62	5.0
163-S	Aug. 16	94:209:340	do	3.0	.78	2.67	.72	1.95	5.9
164-S	do	94:209:340	do	1.5	.68	2.67	.74	1.88	6.1
165-V	do	94:209:340	do	1.2	.68	2.67	.74	1.88	6.1
166-V	do	94:194:360	do	1.4	.68	2.47	.78	1.70	6.0
167-V	do	94:194:360	do	.8	.65	2.47	.79	1.68	6.1
168-V	do	94:177:378	do	1.5	.67	2.27	.82	1.54	6.0
169-V	do	94:177:378	do	.9	.64	2.27	.83	1.52	6.1
170-S	Aug. 17	94:209:340	do	2.9	.75	2.67	.73	1.93	6.0
171-S	do	94:209:340	do	1.4	.66	2.67	.74	1.86	6.1
172-V	do	94:209:340	do	1.4	.66	2.67	.74	1.86	6.1
173-V	do	94:194:360	do	1.6	.66	2.47	.78	1.69	6.1
174-V	do	94:194:360	do	.6	.62	2.47	.79	1.66	6.1
175-V	do	94:177:378	do	1.5	.65	2.27	.82	1.53	6.1
176-V	do	94:177:378	do	.8	.61	2.27	.83	1.50	6.1
177-S	Aug. 22	94:209:340	do	2.6	.80	2.67	.72	1.97	5.9
178-S	do	94:209:340	do	1.1	.70	2.67	.74	1.89	6.0
179-V	do	94:209:340	do	1.1	.70	2.67	.74	1.89	6.0
180-V	do	94:209:391	do	1.9	.81	2.67	.78	1.72	5.5
181-V	do	94:209:417	do	1.4	.81	2.67	.81	1.61	5.4
182-V	do	94:209:443	do	1.0	.81	2.67	.83	1.52	5.2
183-S	Aug. 23	94:176:354	Gravel	2.8	.74	2.24	.72	2.24	5.9
184-S	do	94:176:354	do	1.0	.63	2.24	.73	2.13	6.1
185-V	do	94:176:354	do	1.2	.63	2.24	.73	2.13	6.1
186-V	do	94:163:376	do	1.1	.61	2.08	.77	1.93	6.0
187-V	do	94:163:376	do	.7	.59	2.08	.78	1.91	6.1
188-V	do	94:146:392	do	1.0	.60	1.88	.81	1.75	6.1
189-V	do	94:146:392	do	.7	.58	1.88	.81	1.73	6.1
190-S	Aug. 24	94:176:354	do	2.6	.75	2.24	.71	2.25	5.9
191-S	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1
192-V	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1

[1] Symbols used indicate:
S=Standard finish.
V=Vibrated.
W/C=Water-cement ratio by volume.
a/c=Sand-cement ratio by absolute volume.
b/b_0=Workability factor—ratio of the absolute volume of coarse aggregate in a unit volume of concrete to the absolute volume of coarse aggregate in a unit volume of coarse aggregate (dry and rodded basis).
M/V=Ratio of the absolute volume of mortar in a unit volume of concrete to the volume of the voids in the coarse aggregate in the same unit volume of concrete (dry and rodded basis).

TABLE 3.—*Data on mixes*—Continued

Section no.	Date laid (1934)	Proportions by weight	Coarse aggregate	Slump	W/C	a/c	b/bo	M/V	Cement factor
									Sacks per cubic yard
		Pounds		*Inches*					
193-V	Aug 24	94:163:376	Gravel	1.6	0.66	2.08	.76	1.97	6.0
194-V	do	94:163:376	do	.9	.62	2.08	.77	1.94	6.0
195-V	do	94:146:392	do	1.4	.63	1.88	.80	1.77	6.0
196-V	do	94:146:392	do	.8	.59	1.88	.81	1.74	6.1
197-S	Aug. 27	94:176:354	do	2.8	.76	2.24	.71	2.26	5.9
198-S	do	94:176:354	do	1.4	.68	2.24	.73	2.18	6.0
199-V	do	94-176-354	do	1.4	.68	2.24	.73	2.18	6.0
200-V	do	94:176:409	do	2.4	.76	2.24	.76	1.96	5.5
201-V	do	94:176:436	do	1.6	.76	2.24	.79	1.84	5.3
202-V	do	94:176:463	do	1.4	.76	2.24	.81	1.73	5.2
203-S	Aug. 28	94:176:354	do	3.1	.73	2.24	.72	2.23	6.0
204-S	do	94:176:354	do	1.2	.65	2.24	.73	2.15	6.1
205-V	do	94:176:354	do	1.2	.65	2.24	.73	2.15	6.1
206-V	do	94:176:409	do	2.2	.73	2.24	.77	1.93	5.5
207-V	do	94:176:436	do	2.0	.73	2.24	.79	1.81	5.4
208-V	do	94:176:463	do	1.6	.73	2.24	.82	1.71	5.2
209-S	Aug. 29	94:176:354	do	2.2	.68	2.24	.73	2.18	6.0
210-S	do	94:176:354	do	1.3	.64	2.24	.73	2.14	6.1
211-V	do	94:176:354	do	1.3	.64	2.24	.73	2.14	6.1
212-V	do	94:176:409	do	1.5	.68	2.24	.78	1.89	5.6
213-V	do	94:176:436	do	1.0	.68	2.24	.80	1.77	5.4
214-V	do	94:176:463	do	.9	.68	2.24	.82	1.67	5.2
215-S	Aug. 30	94:176:354	do	2.4	.73	2.24	.72	2.23	6.0
216-S	do	94:176:354	do	1.0	.65	2.24	.73	2.15	6.1
217-V	do	94:176:354	do	1.0	.65	2.24	.73	2.15	6.1
218-V	do	94:176:409	do	1.3	.73	2.24	.77	1.93	5.5
219-V	do	94:176:436	do	1.0	.73	2.24	.79	1.81	5.4
220-V	do	94:176:463	do	.8	.73	2.24	.82	1.71	5.2
221-S	Aug. 31	94:176:354	do	2.4	.72	2.24	.72	2.22	6.0
222-S	do	94:176:354	do	1.3	.64	2.24	.73	2.14	6.1
223-V	do	94:176:354	do	1.2	.64	2.24	.73	2.14	6.1
224-V	do	94:192:409	do	1.4	.72	2.45	.75	2.01	5.4
225-V	do	94:200:436	do	1.0	.72	2.55	.77	1.92	5.2
226-V	do	94:209:463	do	.8	.71	2.67	.79	1.85	5.0
227-S	Sept. 4	94:209:340	Stone	2.5	.74	2.67	.73	1.92	6.0
228-S	do	94:209:340	do	1.2	.66	2.67	.74	1.86	6.1
229-V	do	94:209:340	do	1.3	.66	2.67	.74	1.86	6.1
230-V	do	94:209:391	do	1.9	.74	2.67	.79	1.67	5.6
231-V	do	94:209:417	do	1.7	.74	2.67	.82	1.57	5.4
232-V	do	94:209:443	do	1.4	.74	2.67	.84	1.48	5.3
233-S	Sept. 5	94:209:340	do	2.4	.75	2.67	.73	1.93	6.0
234-S	do	94:209:340	do	1.0	.64	2.67	.75	1.85	6.1
235-V	do	94:209:340	do	1.0	.64	2.67	.75	1.85	6.1
236-V	do	94:209:391	do	1.4	.74	2.67	.79	1.67	5.6
237-V	do	94:209:417	do	1.4	.74	2.67	.82	1.57	5.4
238-V	do	94:209:443	do	1.0	.74	2.67	.84	1.48	5.3
239-S	Sept. 6	94:209:340	do	2.9	.74	2.67	.73	1.92	6.0
240-S	do	94:209:340	do	1.2	.64	2.67	.75	1.85	6.1
241-V	do	94:209:340	do	1.2	.64	2.67	.75	1.85	6.1
242-V	do	94:224:391	do	2.0	.74	2.86	.78	1.73	5.5
243-V	do	94:231:417	do	1.5	.74	2.94	.80	1.65	5.3
244-V	do	94:239:443	do	1.0	.74	3.04	.81	1.58	5.1
245-S	Sept. 10	94:209:340	do	1.4	.71	2.67	.73	1.90	6.0
246-V	do	94:209:340	do	1.7	.71	2.67	.73	1.90	6.0
247-S	do	94:209:340	do	3.1	.77	2.67	.73	1.95	5.9
248-V	do	94:209:391	do	2.6	.77	2.67	.79	1.69	5.6
249-V	do	94:209:417	do	1.8	.77	2.67	.81	1.59	5.4
250-V	do	94:209:443	do	1.8	.77	2.67	.83	1.49	5.2
251-S	Sept. 11	94:209:340	do	2.6	.76	2.67	.73	1.94	5.9
252-S	do	94:209:340	do	1.1	.69	2.67	.74	1.89	6.0
253-V	do	94:209:340	do	1.0	.69	2.67	.74	1.89	6.0
254-V	do	94:209:413	do	1.2	.76	2.67	.84	1.49	5.3
255-S	Sept. 20	94:209:340	do	2.1	.74	2.67	.73	1.92	6.0
256-S	do	94:209:340	do	1.1	.68	2.67	.74	1.88	6.1
257-V	do	94:209:340	do	1.2	.68	2.67	.74	1.88	6.1
258-V	do	94:209:443	do	.8	.74	2.67	.84	1.48	5.3
259-S	Sept. 24	94:209:340	do	2.8	.75	2.67	.73	1.93	6.0
260-S	do	94:209:340	do	1.4	.67	2.67	.74	1.87	6.1
261-V	do	94:209:340	do	1.6	.67	2.67	.74	1.87	6.1
262-V	do	94:209:391	do	2.0	.74	2.67	.79	1.67	5.6
263-V	do	94:209:417	do	1.2	.70	2.67	.82	1.54	5.5
264-V	do	94:209:443	do	.8	.69	2.67	.85	1.45	5.3
265-S	Sept. 25	94:209:340	do	1.6	.72	2.67	.73	1.91	6.0
266-S	do	94:209:340	do	2.3	.75	2.67	.73	1.93	6.0
267-V	do	94:209:340	do	1.6	.72	2.67	.73	1.91	6.0
268-V	do	94:209:391	do	1.6	.75	2.67	.79	1.68	5.6
269-V	do	94:209:417	do	1.4	.75	2.67	.81	1.57	5.4
270-V	do	94:209:443	do	.8	.75	2.67	.84	1.48	5.3

In designating variations of the base mix for vibration, the possibility of effecting economies through cement saving was considered as well as the question of improving quality. In order to obtain the comparisons desired, the base mix was varied in three different ways. In one variation coarse aggregate only was added to the base mix, the water-cement ratio and the sand-cement ratio remaining constant. This resulted in reducing the cement content and slump in proportion to the amount of coarse aggregate added. In another variation the quantities of both fine and coarse aggregate were increased, the water-cement ratio remaining constant and the cement content and slump being still further reduced. In the third variation the cement content was held constant and the water-cement and sand-cement ratios reduced. The mix characteristics of each section as well as the date laid and the method of finishing are shown in table 3.

Referring to table 3, it will be noted that either six or seven sections were constructed during each day's run. The first three included: (1) The base mix, standard finish, at approximately 2½-inch slump; (2) the base mix, standard finish, reduced to about 1-inch slump by lowering the water-cement ratio; and (3) the same mix as (2) except that the concrete was vibrated. The proportions used in the balance of the sections in each day's run were variations of the base mix as indicated in table 3. All of these were vibrated. It will be observed that the base mix, standard finish, in terms of which all of the other sections were rated, was repeated each day. In this way it was possible to eliminate entirely from the comparisons the effect of differences in quality resulting from variable weather conditions during the progress of the work. It is believed that this procedure, although it added considerably to the volume of the work, materially increased its value.

Departing from the practice formerly employed, the concrete was deposited in two layers in a manner similar to that used when mesh reinforcing is to be installed. The longitudinal wooden separators were placed in position after the first batch was deposited and spread to approximately one-half the depth of the section. The second batch was then deposited and spread in the usual way. In series A each bucket was dumped by moving it laterally across the end of the section farthest from the mixer. The concrete was spread with the finishing machine, using a special strike-off for the first layer.

This method of dumping the concrete was criticized as not being comparable to the usual field practice. Consequently, in series B and C, this part of the procedure was changed by dumping each batch in the center of the section at the end farthest from the mixer and using the bucket to spread the concrete toward the mixer. This spread the concrete to a width of approximately 6 feet and was followed by hand shoveling both ways to the forms and with final strike-off by the machine. Figure 1 illustrates the method of depositing the batch used in series A and in series B and C.

STRENGTH AND HONEYCOMBING OF CONCRETE SLABS MEASURED

All of the 7-inch slabs were tested in the field as simple beams with the load applied at the third points of a 54-inch span. Details regarding the apparatus and methods used will be found in earlier reports on the subject.[2] [3] The 10-inch slabs were brought to the laboratory and tested in a 200,000-pound universal testing machine, the required breaking loads being beyond the capacity of the field apparatus. Insofar as the span length and point of application of load were concerned, the same procedure was followed as was used with the 7-inch specimens. All flexure specimens were approximately 9½ months old when tested.

Five cores, 6 inches in diameter, were drilled from each test section, three for density and absorption tests and two for crushing strength tests. Specimens tested for density were drilled from slabs 1, 3, and 5. (See

[2] The Effect of Vibration and Delayed Finishing on Pavement Slabs. PUBLIC ROADS, vol. 14, no. 8, October 1933.

[3] Studies of Paving Concrete, by F. H. Jackson and W. F. Kellerman. PUBLIC ROADS, vol. 12, no. 6, August 1931.

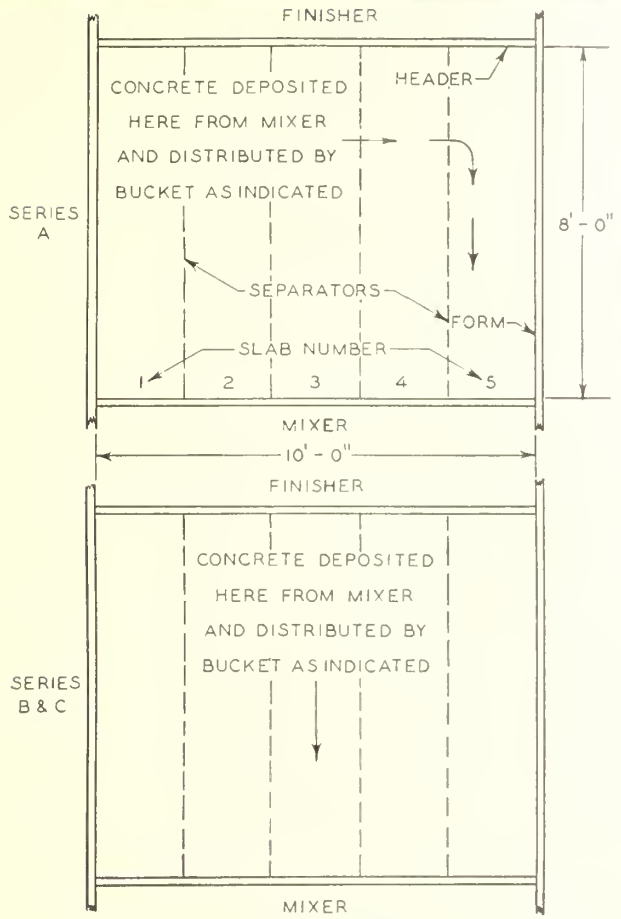


FIGURE 1.—SKETCH SHOWING HOW CONCRETE WAS DEPOSITED ON SUBGRADE BY MIXER DISCHARGE BUCKET.

fig. 1.) In the case of the cores for strength tests, one was drilled from slab 3 and one was drilled from either slab 1 or 5. This was done so that concrete from the center and both sides of the test section would be represented. All cores were drilled from broken slabs, care being taken to select concrete free from honeycomb. Absorption values were computed on the basis of 5 hours' immersion in boiling water. Compression and absorption tests were made when the specimens were approximately 14 months old.

The results of the flexure tests on the individual 24-inch slabs are given in table 4, which shows also the percentage of variation of each value from the average for the section as well as the average variation for the section.

The percentages of honeycomb in the bottom surface and at the cross-section where failure in flexure occurred are given for each test slab in table 5. In general, the same procedure was used for measuring the amount of honeycomb as was employed in the earlier work.[2] For the purpose of determining the rating of each section, an average of the 10 determinations, 5 on the bottom and 5 at the break, was obtained. These are the values which are indicated in the various charts.

Figures 2 to 5, inclusive, illustrate the distribution of honeycomb on the bottom surface of typical sections (series B) representing concrete of medium versus dry consistency, and standard finish versus vibrated.

[2] The Effect of Vibration and Delayed Finishing on Pavement Slabs. PUBLIC ROADS, vol. 14, no. 8, October 1933.

TABLE 4.—*Individual slab strength and uniformity*

Section no.	Modulus of rupture of slab no.—						Variation from average					
	1	2	3	4	5	Average	1	2	3	4	5	Average
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	698	721	703	739	675	707	1.3	2.0	0.6	4.5	4.5	2.6
2	749	751	674	652	650	695	7.8	8.1	3.0	6.2	6.5	6.3
3	676	671	703	659	703	682	.9	1.6	3.1	3.4	3.1	2.4
4	628	607	637	637	708	643	2.3	5.6	.9	.9	10.1	4.0
5	669	712	677	723	701	696	3.9	2.3	2.7	3.9	.7	2.7
6	623	615	635	674	664	642	3.0	4.2	1.1	5.0	3.4	3.3
7	694	769	726	574	662	685	1.3	12.3	6.0	16.2	3.4	7.8
8	658	752	667	684	678	688	4.4	9.3	3.1	.6	1.5	3.8
9	729	750	701	704	672	711	2.5	5.5	1.4	1.0	5.5	3.2
10	625	738	687	643	672	673	7.1	9.7	2.1	4.5	.1	4.7
11	650	699	------	------	------	[1] 674	3.6	3.7	------	------	------	[1] 3.6
12	------	644	627	[2] 486	638	[1] 599	------	7.5	4.7	18.9	6.5	[1] 9.4
13	634	726	636	700	618	663	4.4	9.5	4.1	5.6	6.8	6.1
14	558	[2] 648	670	762	636	655	14.8	1.1	2.3	16.3	2.9	7.5
15	515	604	660	733	[2] 456	594	13.3	1.7	11.1	23.4	23.2	14.5
16	577	676	652	671	657	647	10.8	4.5	.8	3.7	1.5	4.3
17	594	------	685	709	656	[1] 661	10.1	------	3.6	7.3	.8	[1] 5.4
18	[2] 467	[2] 517	610	698	581	575	18.8	10.1	6.1	21.4	1.0	11.5
19	[2] 592	687	687	592	[2] 721	656	9.8	4.7	4.7	9.8	9.9	7.8
20	586	678	628	682	611	637	8.0	6.4	1.4	7.1	4.1	5.4
21	700	[2] 670	[2] 656	698	[2] 680	681	2.8	1.6	3.7	2.5	.1	2.1
22	721	678	723	648	706	695	3.7	2.4	4.0	6.8	1.6	3.7
23	667	710	685	693	707	692	3.6	2.6	1.0	.1	2.2	1.9
24	610	667	639	696	669	656	7.0	1.7	2.6	6.1	2.0	3.9
25	664	615	703	700	747	686	3.2	10.4	2.5	2.0	8.9	5.4
26	547	565	691	643	566	602	9.1	6.1	14.8	6.8	6.0	8.6
27	737	832	778	753	652	750	1.7	10.9	3.7	.4	13.1	6.0
28	722	724	721	742	652	712	1.4	1.7	1.3	4.2	8.4	3.4
29	687	699	748	750	693	715	3.9	2.2	4.6	4.9	3.1	3.7
30	663	709	683	797	709	712	6.9	.4	4.1	11.9	.4	4.7
31	833	890	836	910	936	881	5.4	1.0	5.1	3.3	6.2	4.2
32	851	821	859	997	902	886	3.9	7.3	3.0	12.5	1.8	5.7
33	968	972	976	1,028	1,045	998	3.0	2.6	2.2	3.0	4.7	3.1
34	857	851	890	950	871	884	3.1	3.7	.7	7.5	1.5	3.3
35	804	830	[2] 905	843	957	868	7.4	4.4	4.3	2.9	10.3	5.9
36	799	785	830	913	935	852	6.2	7.9	2.6	7.2	9.7	6.7
37	787	747	717	942	986	836	5.9	10.6	14.2	12.7	17.9	12.3
38	848	875	799	------	984	[1] 876	3.2	.1	8.8	------	12.3	[1] 6.1
39	997	985	979	996	986	989	.8	.4	1.0	.7	.3	.6
40	961	896	908	------	1,043	[1] 952	.9	5.9	4.6	------	9.6	[1] 5.2
41	851	818	811	754	1,008	848	.3	3.5	4.4	11.1	18.9	7.6
42	678	774	796	869	871	798	15.0	3.0	.3	8.9	9.1	7.3
43	897	869	896	903	916	896	.1	3.0	0	.8	2.2	1.2
44	964	946	924	------	838	[1] 918	5.0	3.0	.7	------	8.7	[1] 4.4
45	845	865	945	914	921	898	5.9	3.7	5.2	1.8	2.6	3.8
46	790	862	921	902	958	887	10.9	2.8	3.8	1.7	8.0	5.4
47	960	974	832	940	931	927	3.6	5.1	10.3	1.4	.4	4.2
48	920	909	1,037	889	898	931	1.2	2.4	11.4	4.5	3.5	4.6
49	975	------	986	974	957	[1] 973	.2	------	1.3	.1	1.6	1.8
50	896	925	862	888	899	894	.2	3.5	3.6	.7	.6	1.7
51	927	932	908	886	------	[1] 913	1.5	2.1	.5	3.0	------	[1] 1.8
52	952	745	965	991	928	916	3.9	18.7	5.3	8.2	1.3	7.5
53	929	990	1,001	945	1,002	973	4.5	1.7	2.9	2.9	3.0	3.0
54	1,048	745	1,024	897	890	921	13.8	19.1	11.2	2.6	3.4	10.0
55	913	739	985	896	892	885	3.2	16.5	11.3	1.2	.8	6.6
56	799	1,003	------	857	850	[1] 877	8.9	14.4	------	2.3	3.1	[1] 7.2
57	679	647	------	720	643	[1] 672	1.0	3.7	------	7.1	4.3	[1] 4.0
58	690	799	787	751	734	752	8.2	6.3	4.7	.1	2.4	4.3
59	726	892	768	751	715	770	5.8	15.8	.3	2.5	7.2	6.3
60	780	801	751	756	735	765	2.0	4.7	1.8	1.2	3.9	2.7
61	691	642	780	694	647	691	0	7.1	12.9	.4	6.4	5.4
62	704	679	701	710	697	698	.9	2.7	.4	1.7	.1	1.2
63	669	722	798	745	733	733	8.7	1.5	8.9	1.6	0	4.1
64	[2] 720	735	741	714	719	726	.8	1.2	2.1	1.7	1.0	1.4
65	746	765	737	704	773	745	.1	2.7	1.1	5.5	3.8	2.6
66	758	769	782	777	729	763	.7	.8	2.5	1.8	4.5	2.1
67	753	768	718	755	737	746	.9	3.0	3.8	1.2	1.2	2.0
68	756	731	795	[2] 778	709	754	.3	3.0	5.4	3.2	6.0	3.6
69	791	694	753	781	762	756	4.6	8.2	.4	3.3	.8	3.5
70	712	677	712	683	724	702	1.4	3.6	1.4	2.7	3.1	2.4
71	662	635	716	689	670	674	1.8	5.8	6.2	2.2	.6	3.3
72	699	745	731	728	698	720	2.9	3.5	1.5	1.1	3.1	2.4
73	645	715	763	733	632	698	7.6	2.4	9.3	5.0	9.5	6.8
74	658	636	656	666	676	658	0	3.3	.3	1.2	2.7	1.5
75	641	717	715	691	[2] 571	667	3.9	7.5	7.2	3.6	14.4	7.3
76	619	690	702	702	626	668	7.3	3.3	5.1	5.1	6.3	5.4
77	691	717	776	679	652	703	1.7	2.0	10.4	3.4	7.3	5.0
78	643	774	718	702	651	698	7.9	10.9	2.9	.6	6.7	5.8
79	[2] 664	745	796	715	640	712	6.7	4.6	11.8	.4	10.1	6.7
80	637	700	645	658	670	662	3.8	5.7	2.6	.6	1.2	2.8
81	640	759	679	727	661	693	7.6	9.5	2.0	4.9	4.6	5.7
82	647	715	701	678	636	675	4.1	5.9	3.9	.4	5.8	4.0
83	654	668	639	675	687	665	1.7	.5	3.9	1.5	3.3	2.2
84	718	740	738	736	[2] 738	734	2.2	.8	.5	.3	.5	.9
85	585	761	725	724	709	701	16.5	8.6	3.4	3.3	1.1	6.6
86	650	677	667	713	655	672	3.3	.7	.7	6.1	2.5	2.7
87	598	606	614	615	613	609	1.8	.5	.8	1.0	.7	1.0
88	581	688	690	640	595	639	9.1	7.7	8.0	.2	6.9	6.4
89	792	752	------	852	781	[1] 794	.3	5.3	------	7.3	1.6	[1] 3.6
90	756	879	[2] 877	845	729	817	7.5	7.6	7.3	3.4	10.8	7.3
91	813	817	813	808	809	812	.1	.6	.1	.5	.4	.3
92	799	760	798	759	720	767	4.2	.9	4.0	1.0	6.1	3.2
93	722	901	873	811	764	814	11.3	10.7	7.2	.4	6.1	7.1
94	717	777	831	834	744	781	8.2	.5	6.4	6.8	4.7	5.3
95	673	659	689	669	707	679	.9	2.9	1.5	1.5	4.1	2.2
96	588	785	899	718	[2] 578	714	17.6	9.9	25.9	.6	19.0	14.6
97	680	810	848	840	660	768	11.5	5.5	10.4	9.4	14.1	10.2
98	702	712	786	772	687	732	4.1	2.7	7.4	5.5	6.1	5.2

[1] Average of less than 5 values.
[2] Broke outside middle third.

TABLE 4.—*Individual slab strength and uniformity*—Continued

Section no.	Modulus of rupture of slab no.—						Variation from average					
	1	2	3	4	5	Average	1	2	3	4	5	Average
	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Percent	Percent	Percent	Percent	Percent	Percent
99	649	750	789	799	624	722	10.1	3.9	9.3	10.7	13.6	9.5
100	562	761	839	801	619	716	21.5	6.3	17.2	11.9	13.6	14.1
101	526	568	553	561	560	554	5.1	2.5	.2	1.3	1.1	2.0
102	518	585	583	584	521	558	7.2	4.8	4.5	4.7	6.6	5.6
103	641	648	644	663	542	628	2.1	3.2	2.6	5.6	13.7	5.4
104	[2] 560	641	676	562	607	609	8.0	5.3	11.0	7.7	.3	6.5
105	531	616	624	630	578	596	10.9	3.4	4.7	5.7	3.0	5.5
106	------	614	583	631	503	[1] 583	------	5.3	0	8.2	13.7	[1] 6.8
107	651	626	699	673	663	662	1.7	5.4	5.6	1.7	.2	2.9
108	628	705	705	696	695	686	8.5	2.8	2.8	1.5	1.3	3.4
109	736	725	776	662	675	715	2.9	1.4	8.5	7.4	5.6	5.2
110	652	683	697	669	634	667	2.3	2.4	4.5	.3	5.0	2.9
111	711	707	663	609	650	668	6.4	5.8	.7	8.8	2.7	4.9
112	563	663	644	567	553	598	5.9	10.9	7.7	5.2	7.5	7.4
113	620	694	634	674	576	640	3.1	8.4	.9	5.3	10.0	5.5
114	585	731	729	681	648	675	13.3	8.3	8.0	.9	4.0	6.9
115	645	730	705	813	765	732	11.9	.3	3.7	11.1	4.5	6.3
116	698	660	781	767	668	715	2.4	7.7	9.2	7.3	6.6	6.6
117	637	578	741	744	756	691	7.8	16.4	7.2	7.7	9.4	9.7
118	725	719	747	754	652	719	.8	0	3.9	4.9	9.3	3.8
119	708	766	787	758	[2] 690	742	4.6	3.2	6.1	2.2	7.0	4.6
120	676	680	660	681	660	671	.7	1.3	1.6	1.5	1.6	1.3
121	758	720	746	739	729	738	2.7	2.4	1.1	.1	1.2	1.5
122	687	791	795	787	665	745	7.8	6.2	6.7	5.6	10.7	7.4
123	740	702	704	678	668	698	6.0	6	.9	2.9	4.3	2.9
124	675	765	810	777	680	741	8.9	3.2	9.3	4.9	8.2	6.9
125	742	716	736	779	659	726	2.2	1.4	1.4	7.3	9.2	4.3
126	721	699	797	805	662	737	2.2	5.2	8.1	9.2	10.2	7.0
127	701	617	671	611	711	662	5.9	6.8	1.4	7.7	7.4	5.8
128	649	[2] 659	720	688	[2] 578	659	1.5	0	9.3	4.4	12.3	5.5
129	722	[2] 729	[2] 688	765	689	719	.4	1.4	4.3	6.4	4.2	3.3
130	702	644	631	695	608	656	7.0	1.8	3.8	5.9	7.3	5.2
131	716	625	709	677	711	688	4.1	9.2	3.1	1.6	3.3	4.3
132	683	594	646	669	609	640	6.7	7.2	.9	4.5	4.8	4.8
133	656	625	675	696	689	668	1.8	6.4	1.0	4.2	3.1	3.3
134	602	702	678	629	617	646	6.8	8.7	5.0	2.6	4.5	5.5
135	654	715	747	726	637	696	6.0	2.7	7.3	4.3	8.5	5.8
136	678	725	706	708	645	692	2.0	4.8	2.0	2.3	6.8	3.6
137	683	651	695	670	641	668	2.2	2.5	4.0	.3	4.0	2.6
138	628	624	695	690	635	654	4.0	4.6	6.3	5.5	2.9	4.7
139	747	824	769	782	707	766	2.5	7.6	.4	2.1	7.7	4.1
140	853	909	863	920	829	875	2.5	3.9	1.4	5.1	5.3	3.6
141	869	950	908	906	574	841	3.3	13.0	8.0	7.7	31.7	12.7
142	891	794	[2] 724	728	832	794	12.2	0	8.8	8.3	4.8	6.8
143	869	758	841	850	801	824	5.5	8.0	2.1	3.2	2.8	4.3
144	814	864	845	860	731	823	1.1	5.0	2.7	4.5	11.2	4.9
145	806	847	832	819	894	840	4.0	.8	1.0	2.5	6.4	2.9
146	794	882	925	912	805	864	8.1	2.1	7.1	5.6	6.8	5.9
147	848	888	968	957	878	908	6.6	2.2	6.6	5.4	3.3	4.8
148	833	821	832	840	789	823	1.2	.2	1.1	2.1	4.1	1.7
149	798	842	898	849	752	828	3.6	1.7	8.5	2.5	9.2	5.1
150	802	813	806	838	732	798	.5	1.9	1.0	5.0	8.3	3.3
151	863	959	838	822	859	868	.6	10.5	3.5	5.3	1.0	4.2
152	784	796	818	832	773	801	2.1	.6	2.1	3.9	3.5	2.4
153	864	914	[2] 808	935	793	863	.1	5.9	6.4	8.3	8.1	5.8
154	851	827	750	814	773	803	6.0	3.0	6.6	1.4	3.7	4.1
155	761	848	844	919	737	822	7.4	3.2	2.7	11.8	10.3	7.1
156	717	798	825	818	708	773	7.2	3.2	6.7	5.8	8.4	6.3
157	808	860	810	890	865	847	4.6	1.5	4.4	5.1	2.1	3.5
158	774	918	961	926	741	864	10.4	6.2	11.2	7.2	14.2	9.8
159	984	909	884	901	966	929	5.9	2.2	4.8	3.0	4.0	4.0
160	847	794	813	824	769	809	4.7	1.9	.5	1.9	4.9	2.8
161	820	935	862	960	797	875	6.3	6.9	1.5	9.7	8.9	6.7
162	787	849	935	902	794	853	7.7	.5	9.6	5.7	6.9	6.1
163	859	822	850	815	745	818	5.0	.5	3.9	.4	8.9	3.7
164	900	862	946	981	849	908	.9	5.1	4.2	8.0	6.5	4.9
165	755	962	896	886	865	873	13.5	10.2	2.6	1.5	.9	5.7
166	892	954	979	903	768	899	.8	6.1	8.9	.4	14.6	6.2
167	866	1,001	965	930	888	930	6.9	7.6	3.8	0	4.5	4.6
168	867	968	987	961	791	915	5.2	5.8	7.9	5.0	13.6	7.5
169	924	872	917	954	880	911	1.4	4.3	.7	4.7	2.4	2.7
170	820	799	827	796	760	800	2.5	.1	3.4	.5	5.0	2.3
171	744	869	810	868	821	822	9.5	5.7	1.5	5.6	.1	4.5
172	873	968	912	897	935	917	4.8	5.6	.5	2.2	2.0	3.0
173	951	931	959	1,005	896	948	3	1.8	1.2	6.0	5.5	3.0
174	964	993	996	1,077	874	981	1.7	1.2	1.5	9.8	10.9	5.0
175	954	849	881	935	848	893	6.8	4.9	1.3	4.7	5.0	4.5
176	893	969	949	913	826	910	1.9	6.5	4.3	.3	9.2	4.4
177	831	856	819	827	820	831	0	3.0	1.4	.5	1.3	1.2
178	666	[2] 874	877	846	867	826	19.4	5.8	6.2	2.4	5.0	7.8
179	875	951	911	787	894	884	1.0	7.6	3.1	11.0	1.1	4.8
180	925	758	850	878	797	842	9.9	10.0	.9	4.3	5.3	6.1
181	849	789	873	896	823	846	.4	6.7	3.2	5.9	2.7	3.8
182	751	879	960	868	863	864	13.1	1.7	11.1	.5	1	5.3
183	705	704	690	646	644	678	4.0	3.8	1.8	4.7	5.0	3.9
184	560	694	743	606	522	625	10.4	11.0	18.9	3.0	16.5	12.0
185	700	746	737	724	693	720	2.8	3.6	2.4	.6	3.7	2.6
186	692	744	791	699	645	714	3.1	4.2	10.8	2.1	9.7	6.0
187	742	723	736	601	677	696	6.6	3.9	5.7	13.6	2.7	6.5
188	709	767	748	716	651	718	1.3	6.8	4.2	.3	9.3	4.4
189	749	709	760	707	678	721	3.9	1.7	5.4	1.9	6.0	3.8
190	692	673	656	706	635	672	3.0	.1	2.4	5.1	5.5	3.2
191	620	706	721	718	645	682	9.1	3.5	5.7	5.3	5.4	5.8
192	741	701	712	697	709	712	4.1	1.5	0	2.1	.4	1.6
193	753	699	760	703	665	716	5.2	2.4	6.1	1.8	7.1	4.5
194	705	763	757	715	[2] 660	720	2.1	6.0	5.1	.7	8.3	4.4
195	731	735	734	695	682	715	2.2	2.8	2.7	2.8	4.6	3.0
196	663	708	670	722	662	685	3.2	3.4	2.2	5.4	3.4	3.5
197	663	704	721	682	697	693	4.3	1.6	4.0	1.6	.6	2.4

TABLE 4.—*Individual slab strength and uniformity*—Continued

Section no.	Modulus of rupture of slab no.—						Variation from average					
	1	2	3	4	5	Average	1	2	3	4	5	Average
	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Percent	Percent	Percent	Percent	Percent	Percent
198	710	[2] 799	800	[2] 722	720	750	5.3	6.5	6.7	3.7	4.0	5.2
199	771	774	814	766	726	770	.1	.5	5.7	.5	5.7	2.5
200	[2] 597	650	734	689	657	665	10.2	2.3	10.4	3.6	1.2	5.5
201	637	631	719	700	657	669	4.8	5.7	7.5	4.6	1.8	4.9
202	704	647	692	706	608	671	4.9	3.6	3.1	5.2	9.4	5.2
203	601	613	632	669	612	625	3.8	1.9	1.1	7.0	2.1	3.2
204	496	670	699	704	453	604	17.9	10.9	15.7	16.6	25.0	17.2
205	[2] 578	674	713	610	647	644	10.2	4.7	10.7	5.3	.5	6.3
206	598	659	633	596	618	621	3.7	6.1	1.9	4.0	.5	3.2
207	577	619	625	632	572	605	4.6	2.3	3.3	4.5	5.5	4.0
208	639	610	665	591	584	618	3.4	1.3	7.6	4.4	5.5	4.4
209	728	682	[2] 675	678	691	691	5.4	1.3	2.3	1.9	0	2.2
210	[2] 659	677	758	718	703	703	6.3	3.7	7.8	2.1	0	4.0
211	701	751	736	763	750	740	5.3	1.5	.5	3.1	1.4	2.4
212	694	641	739	706	678	692	.3	7.4	6.8	2.0	2.0	3.7
213	[2] 651	756	731	726	741	721	9.7	4.9	1.4	.7	2.8	3.9
214	645	645	706	700	686	676	4.6	4.6	4.4	3.6	1.5	3.7
215	685	644	657	721	705	682	.4	5.6	3.7	5.7	3.4	3.8
216	[2] 615	742	642	[2] 627	650	655	6.1	13.3	2.0	4.3	.8	5.3
217	704	743	708	709	660	705	.1	5.4	.4	.6	6.4	2.6
218	744	690	669	717	687	701	6.1	1.6	4.6	2.3	2.0	3.3
219	632	556	[2] 692	683	703	653	3.2	14.9	6.0	4.6	7.7	7.3
220	651	692	681	710	703	687	5.2	.7	.9	3.3	2.3	2.5
221	628	704	666	699	718	683	8.1	3.1	2.5	2.3	5.1	4.2
222	626	660	725	683	646	668	6.3	1.2	8.5	2.2	3.3	4.3
223	700	688	680	713	704	697	.4	1.3	2.4	2.3	1.0	1.5
224	667	672	720	690	681	686	2.8	2.0	5.0	.6	.7	2.2
225	[2] 707	684	737	707	617	690	2.5	.9	6.8	2.5	10.6	4.7
226	600	666	682	643	624	643	6.7	3.6	6.1	0	3.0	3.9
227	827	826	[2] 831	814	764	812	1.8	1.7	2.3	.2	5.9	2.4
228	795	859	898	893	860	861	7.7	.2	4.3	3.7	.1	3.2
229	882	824	952	978	848	897	1.7	8.1	6.1	9.0	5.5	6.1
230	873	780	821	814	829	823	6.1	5.2	.2	1.1	.7	2.7
231	829	810	929	882	872	864	4.1	6.3	7.5	2.1	.9	4.2
232	752	799	885	852	797	817	8.0	2.2	8.3	4.3	2.4	5.0
233	916	859	856	915	703	850	7.8	1.1	.7	7.6	17.3	6.9
234	790	921	917	803	751	836	5.5	10.2	9.7	3.9	10.2	7.9
235	843	872	875	818	852	852	1.1	2.3	2.7	4.0	0	2.0
236	879	841	833	822	805	836	5.1	.6	.4	1.7	3.7	2.3
237	764	842	837	902	802	829	7.8	1.6	1.0	8.8	3.3	4.5
238	637	799	943	899	860	828	23.1	3.5	13.9	8.6	3.9	10.6
239	767	825	981	881	822	855	10.3	3.5	14.7	3.0	3.9	7.1
240	753	784	878	823	791	806	6.6	2.7	8.9	2.1	1.9	4.4
241	885	905	956	957	854	911	2.9	.7	4.9	5.0	6.3	4.0
242	853	834	856	868	851	852	.1	2.1	.5	1.9	.1	.9
243	865	794	823	747	737	793	9.1	.1	3.8	5.8	7.1	5.2
244	[2] 742	779	795	821	799	787	5.7	1.0	1.0	4.3	1.5	2.7
245	734	814	756	763	628	739	.7	10.2	2.3	3.2	15.0	6.3
246	703	768	852	714	823	772	8.9	.5	10.4	7.5	6.6	6.8
247	680	810	836	757	813	779	12.7	4.0	7.3	2.8	4.4	6.2
248	803	799	860	932	821	843	4.7	5.2	2.0	10.6	2.6	5.0
249	756	873	871	843	759	820	7.8	6.5	6.2	2.8	7.4	6.1
250	674	799	839	803	[2] 720	767	12.1	4.2	9.4	4.7	6.1	7.3
251	[2] 770	756	810	[2] 825	803	793	2.9	4.7	2.1	4.0	1.3	3.0
252	711	770	793	793	718	757	6.1	1.7	4.8	4.8	5.2	4.5
253	773	[2] 796	796	765	761	778	.6	2.3	2.3	1.7	2.2	1.8
254	732	806	831	773	710	770	4.9	4.7	7.9	.4	7.8	5.1
255	825	851	824	850	864	843	2.1	.9	2.3	.8	2.5	1.7
256	916	874	856	869	[2] 668	837	9.4	4.4	2.3	3.8	20.2	8.0
257	829	843	905	839	731	829	0	1.7	9.2	1.2	11.8	4.8
258	[2] 589	810	941	856	605	760	22.5	6.6	23.8	12.6	20.4	17.2
259	811	744	770	733	820	776	4.5	4.1	.8	5.5	5.7	4.1
260	856	871	824	820	807	836	2.4	4.2	1.4	1.9	3.5	2.7
261	891	878	[2] 895	888	895	889	.2	1.2	.7	.1	.7	.6
262	863	797	908	873	823	853	1.2	6.6	6.4	2.3	3.5	4.0
263	818	872	931	839	836	859	4.8	1.5	8.4	2.3	2.7	3.9
264	809	876	895	855	791	845	4.3	3.7	5.9	1.2	6.4	4.3
265	827	778	893	912	[2] 846	851	2.8	8.6	4.9	7.2	.6	4.8
266	929	902	827	866	865	878	5.8	2.7	5.8	1.4	1.5	3.4
267	879	830	855	832	835	846	3.9	1.9	1.1	1.7	1.3	2.0
268	812	878	843	908	792	847	4.1	3.7	.5	7.2	6.5	4.4
269	790	811	879	895	769	829	4.7	2.2	6.0	8.0	7.2	5.6
270	777	807	870	754	707	783	.8	3.1	11.1	3.7	9.7	5.7

[1] Average of less than 5 values. [2] Broke outside middle third.

Control flexure specimens consisting of four 7 by 7 by 30-inch beams were cast in connection with each standard-finish section up to section no. 151, after which a set of four beams was cast for every section. Three control cylinders, 6- by 12-inch, were cast for each section. None of the control specimens was vibrated. All were job cured and were the same age as the pavement slabs and the cores when tested. All specimens were tested wet. The results are shown in table 6 together with the corresponding average strengths of the pavement slabs and the cores.

Results of tests for bulk specific gravity and absorption are shown in table 7.

TABLE 5.—*Percentage of honeycomb in slabs*

Slab no.	Honeycomb		Slab no.	Honeycomb	
	Bottom	Break		Bottom	Break
	Percent	*Percent*		*Percent*	*Percent*
1-1	17.1	0	15-1	92.4	13.5
2	1.1	0	2	82.3	8.0
3	0	0	3	25.6	0
4	7.4	0	4	34.2	0
5	22.1	0	5	67.7	7.3
Average	9.5	0	Average	60.4	5.8
2-1	7.5	0	16-1	39.8	2.6
2	1.4	0	2	9.4	0
3	2.4	0	3	17.4	0
4	28.6	.9	4	52.0	1.2
5	43.9	.6	5	13.5	2.0
Average	16.8	.3	Average	26.4	1.2
3-1	0	0	17-1	57.3	3.8
2	5.6	0	2	15.5	.3
3	7.0	0	3	7.7	0
4	14.0	0	4	12.7	0
5	42.0	0	5	45.1	.6
Average	13.7	0	Average	27.7	.9
4-1	4.3	0	18-1	61.8	15.4
2	0	0	2	31.0	6.0
3	0	0	3	31.7	.9
4	27.6	2.7	4	29.4	0
5	22.2	0	5	57.3	3.5
Average	10.8	.5	Average	42.8	5.2
5-1	7.2	0	19-1	17.8	4.0
2	4.7	0	2	1.7	0
3	2.8	0	3	15.6	0
4	.8	0	4	7.8	2.3
5	9.6	0	5	9.0	0
Average	5.0	0	Average	10.4	1.3
6-1	1.9	0	20-1	78.4	5.5
2	2.2	0	2	49.4	.9
3	8.3	0	3	52.8	4.4
4	21.0	0.9	4	48.3	.9
5	18.3	0	5	41.7	5.1
Average	10.3	.2	Average	54.1	3.4
7-1	15.9	.5	21-1	14.5	.3
2	0	0	2	2.2	0
3	0	0	3	11.4	.3
4	28.3	.5	4	1.7	0
5	13.4	.5	5	10.3	0
Average	11.5	.3	Average	8.0	.1
8-1	16.9	5.5	22-1	1.0	0
2	0	0	2	1.8	0
3	0	0	3	0	0
4	20.2	3.7	4	2.1	0
5	4.2	0	5	10.1	.3
Average	8.3	1.8	Average	3.0	.1
9-1	0	0	23-1	7.4	.3
2	0	0	2	0	0
3	0	0	3	0	0
4	0	0	4	7.6	0
5	4.2	0	5	9.2	0
Average	.8	0	Average	4.8	.1
10-1	0	0	24-1	17.4	.6
2	2.4	0	2	22.6	.6
3	3.5	0	3	21.2	2.1
4	3.8	0	4	23.8	0
5	3.6	0	5	51.2	.6
Average	2.7	0	Average	27.2	.8
11-1	0	0	25-1	3.9	0
2	0	0	2	7.9	0
3	0	0	3	10.5	0
4	12.3	0	4	15.2	0
5	2.2	0	5	23.9	0
Average	2.9	0	Average	12.3	0
12-1	0	0	26-1	67.6	8.6
2	0	0	2	47.3	4.2
3	2.9	.3	3	31.2	0
4	21.5	0	4	35.3	3.0
5	22.9	0	5	68.0	.3
Average	9.5	.1	Average	49.9	3.2
13-1	34.4	1.5	27-1	4.5	0
2	6.0	0	2	20.5	0
3	15.7	0	3	0	0
4	11.4	2.1	4	6.6	0
5	31.2	3.1	5	13.1	0
Average	19.7	1.3	Average	8.9	0
14-1	71.0	6.9	28-1	6.3	0
2	43.2	5.4	2	0	0
3	48.2	.6	3	0	0
4	59.7	.6	4	10.3	0
5	67.2	3.7	5	5.2	0
Average	57.9	3.4	Average	4.4	0

[1] Not included in average. Slab not tested.

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb		Slab no.	Honeycomb	
	Bottom	Break		Bottom	Break
	Percent	*Percent*		*Percent*	*Percent*
29-1	20.2	2.4	43-1	7.4	0
2	4.7	0	2	10.8	0
3	1.9	0	3	3.0	0
4	13.1	0	4	14.8	0
5	5.1	0	5	8.7	1.2
Average	9.0	.5	Average	8.9	.2
30-1	33.8	4.1	44-1	29.5	0
2	4.4	0	2	13.8	0
3	4.4	0	3	0	0
4	16.1	0	4	36.7	3.0
5	25.3	0	5	39.5	4.9
Average	16.8	.8	Average	23.9	1.6
31-1	36.7	2.7	45-1	7.6	.3
2	5.6	0	2	12.8	.6
3	12.9	0	3	0	0
4	22.5	0	4	6.7	.6
5	21.0	0	5	0	0
Average	19.7	.5	Average	5.4	.3
32-1	42.8	8.7	46-1	7.5	0
2	16.0	5.8	2	17.7	0
3	38.2	1.1	3	3.5	0
4	17.9	0	4	2.4	0
5	17.5	.6	5	3.6	0
Average	26.5	3.2	Average	6.9	0
33-1	0	0	47-1	2.1	0
2	0	0	2	63.9	1.2
3	0	0	3	24.3	0
4	0	0	4	27.1	0
5	0	0	5	39.6	.9
Average	0	0	Average	31.4	.4
34-1	0	0	48-1	4.6	0
2	31.7	.6	2	36.8	0
3	18.3	.6	3	11.4	0
4	22.6	0	4	27.9	0
5	0	0	5	13.5	0
Average	14.5	.2	Average	18.8	0
35-1	35.4	.6	49-1	22.2	.6
2	11.8	1.2	2	24.6	1.2
3	26.9	0	3	8.8	.3
4	10.8	0	4	21.2	.6
5	5.5	0	5	35.6	1.2
Average	18.1	.4	Average	22.5	.8
36-1	16.5	0	50-1	27.9	1.2
2	57.8	5.1	2	3.8	0
3	37.4	.9	3	21.9	0
4	8.3	0	4	16.9	0
5	13.1	0	5	21.7	.9
Average	26.6	1.2	Average	18.4	.4
37-1	33.9	4.7	51-1	26.4	.6
2	14.9	9.9	2	11.1	.6
3	17.2	11.0	3	29.5	.6
4	23.9	0	4	22.2	.6
5	22.5	.6	5	[1] 26.0	------
Average	22.5	5.2	Average	[2] 22.3	[2] .6
38-1	39.7	7.0	52-1	2.8	0
2	9.9	1.2	2	4.1	0
3	14.8	5.1	3	18.0	.3
4	[1] 29.2	------	4	17.3	.6
5	38.1	2.3	5	15.6	.6
Average	[2] 25.6	[2] 3.9	Average	11.6	.3
39-1	1.5	0	53-1	15.6	.9
2	0	0	2	0	0
3	0	0	3	3.2	0
4	0	0	4	2.1	0
5	1.4	0	5	5.0	0
Average	.6	0	Average	5.2	.2
40-1	13.8	3.0	54-1	8.3	0
2	11.9	0	2	24.7	1.2
3	1.9	.6	3	0	0
4	12.5	0	4	15.2	0
5	1.7	0	5	16.2	0
Average	8.4	.7	Average	12.9	.2
41-1	23.2	.6	55-1	13.8	0
2	19.4	3.9	2	5.3	0
3	5.1	0	3	6.2	0
4	18.4	0	4	29.0	.6
5	3.6	0	5	5.1	0
Average	13.9	.9	Average	11.9	.1
42-1	31.2	9.7	56-1	23.6	.6
2	40.5	2.4	2	48.3	1.2
3	13.5	.6	3	18.1	.9
4	25.1	3.0	4	35.6	.6
5	15.2	0	5	28.2	.6
Average	25.1	3.1	Average	30.8	.8

[2] Average of 4 values.

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
57-1	3.4	0
2	26.6	2.1
3	44.1	
4	32.6	1.4
5	18.2	0
Average	[2] 20.2	2.9
58-1	54.2	2.1
2	25.6	.9
3	15.6	0
4	13.4	0
5	32.2	.6
Average	28.2	.7
59-1	2.1	0
2	10.2	0
3	6.5	0
4	3.2	0
5	18.3	0
Average	8.1	0
60-1	16.2	.6
2	4.0	0
3	11.6	0
4	3.7	0
5	7.1	0
Average	8.5	.1
61-1	13.8	.6
2	30.6	.6
3	10.4	.6
4	24.1	1.4
5	39.0	3.4
Average	23.6	1.3
62-1	7.2	0
2	12.0	.6
3	29.7	.6
4	16.2	0
5	15.2	0
Average	16.1	.2
63-1	62.4	4.4
2	74.0	2.9
3	17.6	.6
4	22.6	.6
5	36.0	1.2
Average	42.5	1.9
64-1	3.8	0
2	2.9	0
3	7.6	0
4	24.7	.6
5	9.8	0
Average	9.8	.1
65-1	14.9	0
2	10.2	.6
3	2.0	0
4	7.9	0
5	41.7	.6
Average	15.3	.2
66-1	1.0	0
2	1.6	0
3	.8	0
4	.8	0
5	4.3	0
Average	1.7	0
67-1	4.7	0
2	0	0
3	3.3	0
4	2.4	0
5	1.5	0
Average	2.4	0
68-1	0	0
2	3.4	0
3	0	0
4	6.3	0
5	25.0	.6
Average	6.9	.1
69-1	0	0
2	0	0
3	0	0
4	0	0
5	4.9	0
Average	1.0	0
70-1	15.6	.6
2	10.9	0
3	0	0
4	21.7	0
5	26.5	.6
Average	14.9	.2
71-1	25.1	3.9
2	5.6	0
3	8.8	0
4	10.0	.6
5	27.1	1.2
Average	15.3	1.1
72-1	55.0	2.7
2	17.2	1.5
3	6.0	0
4	9.3	0
5	40.5	2.0
Average	25.6	1.2
73-1	19.4	2.0
2	7.9	0
3	0	0
4	3.3	0
5	10.2	.6
Average	8.2	.5
74-1	9.5	.3
2	8.1	0
3	18.7	1.7
4	11.7	0
5	11.6	0
Average	11.9	.4
75-1	47.9	1.8
2	8.1	0
3	0	0
4	.7	0
5	42.3	3.9
Average	19.8	1.1
76-1	47.7	4.6
2	9.8	1.5
3	0	0
4	25.6	.6
5	42.9	1.7
Average	25.2	1.7
77-1	11.8	.6
2	5.1	0
3	12.3	0
4	3.8	0
5	17.2	.6
Average	10.0	.2
78-1	12.9	.6
2	2.8	0
3	.7	0
4	7.5	0
5	8.5	0
Average	6.5	.1
79-1	9.8	.9
2	12.8	0
3	.6	0
4	8.9	0
5	0	0
Average	6.4	.2
80-1	14.6	0
2	0	0
3	0	0
4	3.7	0
5	0	0
Average	3.7	0
81-1	37.6	1.7
2	0	0
3	8.8	0
4	6.9	0
5	12.1	0
Average	13.1	.3
82-1	38.2	.6
2	4.6	.3
3	0	0
4	0	0
5	21.5	1.2
Average	12.9	.4
83-1	3.6	.6
2	4.0	0
3	0	0
4	3.5	0
5	11.0	.6
Average	4.4	.2
84-1	21.9	.7
2	0	0
3	0	0
4	4.9	0
5	11.7	.3
Average	7.7	.4

[1] Not included in average. Slab not tested.

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
85-1	38.3	6.4
2	0	0
3	0	0
4	0	0
5	0	0
Average	7.7	1.3
86-1	18.8	.6
2	0	0
3	0	0
4	2.2	0
5	7.5	.6
Average	5.7	.2
87-1	4.6	0
2	0	0
3	0	0
4	0	0
5	8.6	0
Average	2.6	0
88-1	33.6	3.9
2	3.1	0
3	0	0
4	6.2	.6
5	23.1	1.5
Average	13.2	1.2
89-1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
90-1	12.2	3.0
2	0	0
3	0	0
4	.8	0
5	26.5	2.3
Average	7.9	1.1
91-1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
92-1	0	0
2	0	0
3	0	0
4	0	0
5	1.7	0
Average	.3	0
93-1	3.3	.6
2	0	0
3	0	0
4	0	0
5	0	0
Average	.7	.1
94-1	28.0	6.4
2	16.8	3.3
3	0	0
4	8.6	.6
5	60.0	5.2
Average	22.7	3.1
95-1	7.0	0
2	0	0
3	0	0
4	0	0
5	3.3	0
Average	2.1	0
96-1	31.7	10.3
2	0	0
3	0	0
4	14.9	1.3
5	15.4	5.6
Average	12.4	3.4
97-1	25.9	5.6
2	4.5	1.7
3	0	0
4	0	0
5	21.5	1.9
Average	10.4	1.8
98-1	14.4	3.5
2	0	0
3	0	0
4	0	0
5	3.5	.7
Average	3.6	.8
99-1	8.3	2.9
2	0	0
3	0	0
4	0	0
5	2.3	0
Average	2.1	.6
100-1	39.0	14.1
2	5.6	0
3	0	0
4	3.8	0
5	48.3	7.6
Average	19.3	4.3
101-1	4.0	0
2	0	0
3	0	0
4	0	0
5	9.6	1.8
Average	2.7	.4
102-1	60.0	10.0
2	31.7	4.0
3	16.9	0
4	28.1	4.9
5	62.6	10.7
Average	39.9	5.9
103-1	28.3	1.1
2	18.4	1.5
3	0	0
4	13.3	0
5	66.5	9.1
Average	25.3	2.3
104-1	8.8	1.2
2	0	0
3	0	0
4	0	0
5	14.5	0
Average	4.7	.2
105-1	30.1	5.7
2	0	0
3	0	0
4	4.4	.8
5	13.5	0
Average	9.6	1.3
106-1	[1] 62.2	[1] 7.9
2	8.8	0
3	0	0
4	8.5	1.1
5	38.5	4.2
Average	[2] 14.0	[2] 1.3
107-1	14.0	1.2
2	6.7	0
3	6.5	0
4	7.1	.6
5	7.1	.6
Average	8.3	.5
108-1	24.4	2.9
2	0	0
3	0	0
4	0	0
5	26.7	2.1
Average	10.2	1.0
109-1	2.6	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	.5	0
110-1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
111-1	0	0
2	2.8	0
3	0	0
4	0	0
5	6.9	0
Average	1.9	0
112-1	5.3	1.2
2	3.7	0
3	0	0
4	6.1	0
5	11.0	.3
Average	5.2	.3

[2] Average of 4 values.

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
113–1	25.1	1.2
2	5.8	0
3	0	0
4	0	0
5	19.7	.6
Average	10.1	.4
114–1	69.9	7.1
2	11.4	0
3	2.8	0
4	15.6	2.1
5	32.6	2.4
Average	26.5	2.3
115–1	17.9	1.2
2	20.3	.6
3	0	0
4	3.0	6
5	13.3	0
Average	10.9	.5
116–1	3.8	0
2	3.5	.3
3	3.8	0
4	0	0
5	2.8	0
Average	2.8	.1
117–1	41.3	1.7
2	22.0	.6
3	0	0
4	10.6	.6
5	15.7	.6
Average	17.9	.7
118–1	2.3	0
2	8.1	0
3	0	0
4	4.4	0
5	22.7	0
Average	7.5	0
119–1	22.5	.6
2	6.5	0
3	0	0
4	12.8	.6
5	30.5	2.4
Average	14.5	.7
120–1	8.8	.6
2	4.1	0
3	5.8	1.5
4	9.4	1.4
5	6.7	.6
Average	7.0	.8
121–1	35.3	.6
2	8.4	0
3	.6	0
4	4.6	0
5	56.6	4.4
Average	21.1	1.0
122–1	6.6	.6
2	2.2	.6
3	1.8	0
4	0	0
5	6.7	.6
Average	3.5	.4
123–1	3.3	0
2	5.8	0
3	0	0
4	0	0
5	2.1	0
Average	2.2	0
124–1	14.3	.9
2	0	0
3	0	0
4	10.2	.3
5	15.0	.6
Average	7.9	.4
125–1	2.8	0
2	1.1	0
3	3.2	0
4	0	0
5	6.5	0
Average	2.7	0
126–1	8.1	.9
2	9.8	0
3	0	0
4	3.8	0
5	66.3	3.1
Average	17.6	.8
127–1	6.5	0
2	0	0
3	0	0
4	1.1	0
5	17.1	.9
Average	4.9	.2
128–1	31.8	1.2
2	3.1	.6
3	1.6	0
4	.9	0
5	18.7	3.3
Average	17.2	1.0
129–1	4.8	0
2	1.0	0
3	6.1	0
4	2.6	0
5	16.8	.3
Average	6.3	.1
130–1	7.3	.3
2	4.9	2.3
3	5.8	.6
4	0	0
5	3.7	0
Average	4.3	.6
131–1	.6	0
2	3.2	0
3	2.0	0
4	0	0
5	2.8	0
Average	1.7	0
132–1	0	0
2	12.6	.9
3	8.3	.6
4	2.6	0
5	3.4	0
Average	5.4	.3
133–1	15.2	.6
2	15.1	3.4
3	5.9	.6
4	9.8	0
5	15.1	1.7
Average	12.2	1.3
134–1	35.9	2.0
2	9.1	0
3	14.0	0
4	23.0	0
5	22.2	.3
Average	20.8	.5
135–1	1.2	0
2	0	0
3	0	0
4	1.5	0
5	3.0	0
Average	1.1	0
136–1	1.3	0
2	2.6	0
3	0	0
4	5.0	.3
5	6.1	0
Average	3.0	.1
137–1	5.1	.3
2	0	0
3	0	0
4	9.4	.6
5	20.3	.9
Average	7.0	.4
138–1	33.3	2.6
2	2.3	0
3	0	0
4	13.9	.3
5	22.1	.6
Average	14.3	.7
139–1	4.0	2.3
2	2.2	0
3	8.5	0
4	5.6	0
5	6.6	0
Average	5.4	.5
140–1	20.1	2.7
2	1.2	0
3	.9	.3
4	3.7	0
5	45.5	4.1
Average	14.3	1.4

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
141–1	16.5	1.5
2	4.4	0
3	0	0
4	6.8	.3
5	21.9	.3
Average	9.9	.4
142–1	4.8	0
2	7.1	0
3	3.1	0
4	9.8	.3
5	1.0	0
Average	5.2	.1
143–1	9.8	.6
2	0	0
3	12.1	0
4	6.2	.3
5	6.0	6.2
Average	6.8	1.4
144–1	8.5	0
2	0	0
3	0	0
4	5.4	.6
5	30.9	3.8
Average	9.0	.9
145–1	19.0	0
2	0	0
3	.9	0
4	1.6	0
5	17.5	.3
Average	7.8	.1
146–1	73.3	5.2
2	5.3	0
3	1.4	0
4	36.2	.9
5	75.5	8.6
Average	38.3	2.9
147–1	4.9	.9
2	0	0
3	0	0
4	2.8	0
5	6.9	0
Average	2.9	.2
148–1	.8	0
2	0	0
3	0	0
4	2.2	0
5	0	0
Average	.6	0
149–1	14.7	.3
2	3.4	0
3	0	0
4	5.8	0
5	.7	0
Average	4.9	.1
150–1	23.0	0
2	6.1	0
3	0	0
4	5.0	0
5	25.3	.9
Average	11.9	.2
151–1	20.8	0
2	0	0
3	1.0	0
4	7.0	0
5	9.0	.6
Average	7.6	.1
152–1	62.2	6.4
2	4.3	.3
3	5.3	.6
4	22.5	3.7
5	55.1	4.3
Average	29.9	3.1
153–1	7.6	.6
2	1.2	0
3	0	0
4	2.2	0
5	28.0	4.5
Average	7.8	1.0
154–1	.3	0
2	.8	0
3	.8	0
4	0	0
5	3.1	0
Average	1.0	0
155–1	10.5	0
2	1.4	0
3	0	0
4	9.7	0
5	50.9	3.3
Average	14.5	.7
156–1	16.7	2.4
2	15.6	4.2
3	1.4	0
4	9.6	.3
5	85.2	7.2
Average	25.7	2.8
157–1	25.8	.6
2	2.6	.3
3	1.0	0
4	3.0	0
5	3.8	0
Average	7.2	.2
158–1	64.9	6.8
2	23.3	.9
3	0	0
4	25.7	1.8
5	75.5	6.5
Average	37.9	3.2
159–1	2.6	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	.5	0
160–1	0	0
2	0	0
3	0	0
4	.6	0
5	1.5	0
Average	.4	0
161–1	3.8	0
2	3.3	0
3	0	0
4	4.7	0
5	1.2	0
Average	2.6	0
162–1	27.1	3.5
2	23.1	2.4
3	0	0
4	6.0	0
5	29.0	1.7
Average	17.0	1.5
163–1	4.9	.9
2	0	0
3	0	0
4	15.6	0
5	5.7	0
Average	5.2	.2
164–1	44.8	0.9
2	5.8	0
3	2.8	0
4	9.4	.6
5	60.0	2.9
Average	24.6	.9
165–1	0	0
2	0	0
3	0	0
4	6.9	0
5	7.8	0
Average	2.9	0
166–1	3.7	0
2	0	0
3	0	0
4	2.1	0
5	28.9	.3
Average	6.9	.1
167–1	6.3	0
2	4.6	0
3	0	0
4	1.9	0
5	16.0	.6
Average	5.8	.1
168–1	2.6	.3
2	3.2	0
3	1.4	0
4	1.0	0
5	3.2	0
Average	2.3	.1

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
169–1	19.6	0.9
2	.9	0
3	0	0
4	17.1	0
5	.5	0
Average	7.6	.2
170–1	1.4	0
2	0	0
3	0	0
4	0	0
5	18.5	0
Average	4.0	0
171–1	34.6	4.2
2	1.0	0
3	0	0
4	13.3	0
5	68.3	.9
Average	23.4	1.0
172–1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
173–1	2.0	0
2	0	0
3	0	0
4	0	0
5	.8	0
Average	.6	0
174–1	22.0	.6
2	4.2	0
3	0	0
4	15.9	0
5	36.1	.9
Average	15.6	.3
175–1	2.4	0
2	3.6	.3
3	1.1	0
4	1.7	0
5	14.6	.6
Average	4.7	.2
176–1	12.5	0
2	6.5	0
3	0	0
4	10.2	.3
5	34.9	1.2
Average	12.8	.3
177–1	7.2	.9
2	11.7	0
3	3.1	0
4	8.2	0
5	3.5	0
Average	6.7	.2
178–1	32.4	13.0
2	12.5	3.7
3	0	0
4	21.7	5.1
5	40.6	4.0
Average	21.4	5.2
179–1	1.7	0
2	0	0
3	0	0
4	2.3	0
5	5.8	.9
Average	2.0	.2
180–1	3.9	.8
2	5.7	0
3	9.8	0
4	3.1	0
5	.8	0
Average	4.7	.2
181–1	13.3	0
2	3.8	0
3	1.6	0
4	3.5	0
5	9.0	0
Average	6.2	0
182–1	15.8	2.4
2	4.8	0
3	1.2	0
4	16.8	.6
5	23.5	0
Average	12.4	.6
183–1	19.0	.9
2	8.1	0
3	2.1	0
4	2.0	0
5	27.6	3.3
Average	11.8	.8

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
184–1	100.0	19.1
2	53.5	3.1
3	29.7	0
4	53.9	12.9
5	100.0	12.1
Average	67.4	9.4
185–1	2.4	0
2	.6	0
3	0	0
4	25.8	.9
5	35.7	1.2
Average	12.9	.4
186–1	10.0	.9
2	1.1	0
3	1.2	0
4	35.1	2.7
5	29.4	.6
Average	15.4	.8
187–1	38.0	3.0
2	24.4	.6
3	37.4	2.1
4	100.0	8.0
5	63.5	5.6
Average	52.7	3.9
188–1	10.1	.6
2	2.3	0
3	3.2	0
4	37.2	2.0
5	27.4	.9
Average	16.0	.7
189–1	33.8	.3
2	9.8	.6
3	17.6	.6
4	81.5	3.9
5	68.1	3.2
Average	42.2	1.7
190–1	21.1	.9
2	8.1	.6
3	5.6	.9
4	5.7	0
5	19.5	.6
Average	12.0	.6
191–1	67.5	4.6
2	4.4	0
3	5.0	0
4	31.8	1.8
5	95.3	4.8
Average	40.8	2.2
192–1	6.1	0
2	29.8	3.0
3	0	0
4	4.4	.3
5	3.5	0
Average	8.8	.7
193–1	4.1	0
2	35.3	1.7
3	2.4	0
4	1.9	0
5	3.8	0
Average	9.5	.3
194–1	11.2	0
2	4.0	0
3	0	0
4	28.3	1.8
5	36.1	2.1
Average	15.9	.8
195–1	8.9	0
2	1.9	0
3	4.3	0
4	2.9	0
5	15.2	.6
Average	6.6	.1
196–1	25.1	2.0
2	4.2	.3
3	0	0
4	35.1	1.7
5	42.7	2.4
Average	21.4	1.3
197–1	45.4	2.7
2	9.0	0
3	1.5	0
4	30.3	3.4
5	18.3	.3
Average	20.9	1.3
198–1	34.0	.9
2	7.1	.6
3	4.0	0
4	22.4	.6
5	84.7	3.2
Average	30.4	1.1

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
199–1	3.4	0
2	1.3	0
3	0	0
4	2.2	0
5	4.4	0
Average	2.3	0
200–1	4.1	0
2	17.3	2.0
3	0	0
4	10.0	0
5	0	0
Average	6.3	.4
201–1	1.4	0
2	14.1	0
3	13.4	.9
4	6.8	1.5
5	7.8	0
Average	8.7	.5
202–1	9.7	.6
2	10.1	1.4
3	19.0	2.3
4	12.8	.9
5	7.1	.6
Average	11.7	1.2
203–1	17.0	.4
2	0	0
3	0	0
4	0	0
5	26.2	.4
Average	8.6	.2
204–1	73.2	9.0
2	17.6	.4
3	3.7	0
4	25.0	.8
5	76.3	9.2
Average	39.2	3.9
205–1	0	0
2	0	0
3	0	0
4	11.3	0
5	13.1	0
Average	4.9	0
206–1	0	0
2	0	0
3	.8	0
4	1.5	0
5	0	0
Average	.5	0
207–1	0	0
2	0	0
3	0	0
4	0	0
5	11.5	0
Average	2.3	0
208–1	3.8	0
2	2.5	0
3	5.2	0
4	11.8	0
5	14.2	0
Average	7.5	0
209–1	8.6	.9
2	16.9	1.4
3	9.2	0
4	11.0	.3
5	14.1	1.5
Average	12.0	.8
210–1	50.5	3.9
2	7.6	.8
3	28.3	.3
4	15.1	.5
5	72.0	4.3
Average	34.7	2.0
211–1	12.2	1.4
2	5.2	.3
3	9.6	1.7
4	9.2	.3
5	4.4	.3
Average	8.1	.8
212–1	6.0	.9
2	9.2	.6
3	12.8	1.2
4	10.4	1.5
5	3.5	.5
Average	8.4	.9
213–1	14.4	.9
2	4.2	.3
3	12.6	1.8
4	19.4	2.4
5	11.2	.8
Average	12.4	1.2

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
214–1	24.6	1.2
2	17.2	.6
3	10.3	1.5
4	26.5	1.6
5	19.4	1.6
Average	19.6	1.3
215–1	16.9	2.0
2	7.2	0
3	12.5	0
4	17.5	0
5	3.1	0
Average	11.4	.4
216–1	96.1	15.5
2	64.7	1.7
3	64.9	2.3
4	77.5	3.7
5	51.4	6.1
Average	70.9	5.9
217–1	14.3	.9
2	15.5	.6
3	1.0	0
4	19.4	0
5	21.2	.3
Average	14.3	.4
218–1	4.2	0
2	17.6	.6
3	2.2	0
4	5.2	0
5	22.6	.6
Average	10.4	.2
219–1	14.4	0
2	6.0	0
3	0	0
4	9.1	.6
5	13.8	0
Average	8.7	.1
220–1	32.1	1.2
2	11.9	0
3	0	0
4	27.4	.6
5	30.4	2.0
Average	20.4	.8
221–1	12.3	.6
2	10.8	0
3	4.5	1.5
4	25.4	1.2
5	26.5	1.5
Average	15.9	1.0
222–1	77.4	4.9
2	14.6	1.8
3	14.4	.6
4	18.4	2.1
5	46.8	3.6
Average	34.3	2.6
223–1	12.2	0
2	5.9	0
3	9.7	.9
4	16.2	0
5	7.7	0
Average	10.3	.2
224–1	4.9	.3
2	13.0	0
3	4.6	0
4	14.4	.6
5	4.9	.9
Average	8.4	.4
225–1	6.1	0
2	3.3	0
3	1.9	0
4	10.3	.6
5	16.8	0
Average	7.7	.1
226–1	20.6	.9
2	18.0	1.2
3	3.9	0
4	24.6	.9
5	53.1	3.4
Average	24.0	1.3
227–1	31.7	2.4
2	8.1	0
3	19.2	.9
4	0	0
5	21.6	3.3
Average	16.1	1.3
228–1	49.2	4.4
2	12.3	0
3	2.0	0
4	5.0	0
5	48.7	2.3
Average	23.4	1.3

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
229–1	12.2	0
2	1.4	0
3	9.2	0
4	4.4	1.2
5	4.0	1.2
Average	6.2	.5
230–1	3.3	0
2	6.9	0
3	20.0	1.2
4	4.1	0
5	.6	0
Average	7.0	.2
231–1	5.4	0
2	6.7	.9
3	19.5	1.2
4	10.6	0
5	1.4	0
Average	8.7	.4
232–1	8.2	.6
2	4.9	0
3	59.1	2.4
4	48.0	1.7
5	30.4	.9
Average	30.1	1.1
233–1	17.5	.6
2	19.6	1.7
3	4.7	.3
4	10.8	0
5	42.8	4.7
Average	19.1	1.5
234–1	89.0	6.1
2	45.7	1.2
3	40.3	.6
4	70.4	5.3
5	90.0	8.3
Average	67.1	4.3
235–1	5.3	.3
2	4.0	0
3	1.1	0
4	22.6	.3
5	21.5	.3
Average	10.9	.2
236–1	4.2	0
2	2.8	0
3	6.1	0
4	14.8	.3
5	23.5	.9
Average	10.3	.2
237–1	32.6	.9
2	25.1	.6
3	7.2	0
4	12.3	0
5	13.2	0
Average	18.1	.3
238–1	39.8	1.8
2	40.1	2.4
3	14.0	0
4	35.3	.6
5	46.9	2.6
Average	35.2	1.5
239–1	22.9	2.9
2	11.8	0
3	3.6	0
4	8.2	.8
5	24.4	3.2
Average	14.2	1.4
240–1	91.1	11.8
2	41.5	9.4
3	15.3	0
4	40.9	7.9
5	95.3	13.0
Average	56.8	8.4
241–1	6.2	0
2	5.3	0
3	1.7	0
4	21.9	.3
5	17.7	.6
Average	10.6	.2
242–1	3.5	0
2	4.6	0
3	4.3	0
4	13.7	0
5	3.3	0
Average	5.9	0
243–1	5.2	.3
2	4.5	0
3	1.7	0
4	21.9	.6
5	10.9	.6
Average	8.8	.3
244–1	5.7	0
2	24.6	.6
3	7.1	0
4	36.4	.3
5	30.1	.6
Average	20.8	.3
245–1	46.3	2.1
2	2.6	.2
3	1.6	0
4	13.3	.2
5	42.9	3.2
Average	21.3	1.1
246–1	0	0
2	0	0
3	1.8	0
4	0	0
5	0	0
Average	.4	0
247–1	30.8	3.5
2	1.7	0
3	0	0
4	0	0
5	20.3	.6
Average	10.6	.8
248–1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
249–1	1.6	0
2	0	0
3	0	0
4	0	0
5	25.6	.2
Average	5.4	0
250–1	0	0
2	4.2	0
3	1.2	0
4	1.2	0
5	24.9	.6
Average	6.3	.1
251–1	14.0	.3
2	0	0
3	0	0
4	0	0
5	2.2	0
Average	3.2	.1
252–1	48.7	8.4
2	24.4	6.0
3	1.9	0
4	17.1	2.9
5	49.3	7.4
Average	28.3	4.9
253–1	5.3	0
2	22.0	0
3	1.4	0
4	0	0
5	0	0
Average	5.7	0
254–1	5.3	0
2	11.6	.6
3	2.4	0
4	5.3	0
5	52.3	.9
Average	15.4	.3
255–1	14.4	0
2	4.0	0
3	10.3	0
4	15.7	0
5	20.8	.3
Average	13.0	.1
256–1	55.7	3.7
2	24.6	2.4
3	10.3	.6
4	38.9	3.9
5	88.0	8.5
Average	43.5	3.8
257–1	20.5	2.4
2	18.5	1.2
3	6.7	0
4	30.8	.9
5	63.9	5.7
Average	28.1	2.0
258–1	81.1	14.5
2	63.3	8.5
3	31.3	1.2
4	54.9	8.2
5	100.0	7.9
Average	66.1	8.0

TABLE 5.—*Percentage of honeycomb in slabs*—Continued

Slab no.	Honeycomb	
	Bottom	Break
	Percent	*Percent*
259–1	13.2	0
2	15.2	0
3	23.3	1.2
4	8.2	0
5	15.1	0
Average	15.0	.2
260–1	26.6	.9
2	3.4	0
3	10.6	0
4	17.2	.9
5	39.6	.6
Average	19.5	.5
261–1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
262–1	5.3	0
2	0	0
3	0	0
4	0	0
5	9.4	0
Average	2.9	0
263–1	2.6	0
2	0	0
3	0	0
4	0	0
5	1.2	0
Average	.8	0
264–1	11.8	.9
2	7.8	.6
3	0	0
4	6.4	.6
5	20.6	.9
Average	9.3	.6
265–1	18.5	0.6
2	17.3	2.0
3	0	0
4	1.5	0
5	55.3	4.2
Average	18.5	1.4
266–1	7.4	0
2	0	0
3	0	0
4	0	0
5	11.1	0
Average	3.7	0
267–1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
Average	0	0
268–1	4.5	0
2	1.6	0
3	0	0
4	.6	0
5	6.4	0
Average	2.6	0
269–1	8.3	0
2	4.9	0
3	0	0
4	0	0
5	1.3	0
Average	2.9	0
270–1	0	0
2	0	0
3	0	0
4	6.0	.6
5	10.6	2.3
Average	3.3	.6

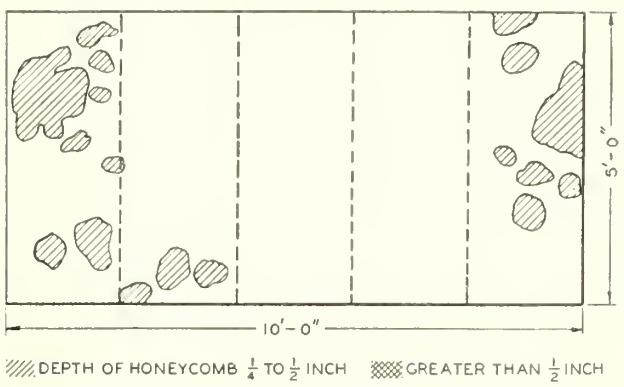


FIGURE 2.—DISTRIBUTION OF HONEYCOMB AT BOTTOM OF A SECTION (SERIES B) HAVING: BASE MIX; STANDARD FINISH; A SLUMP OF 2.5 INCHES; AND 10.1 PERCENT HONEYCOMB.

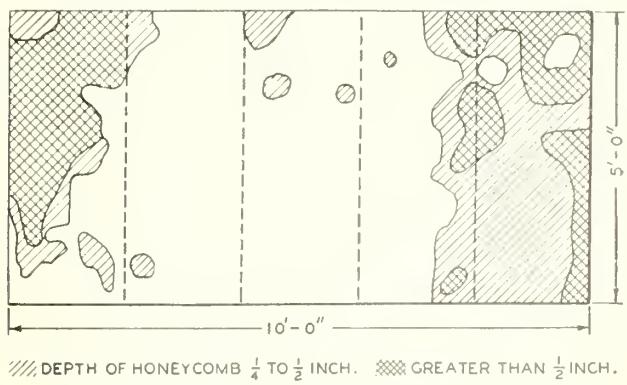


FIGURE 3.—DISTRIBUTION OF HONEYCOMB AT BOTTOM OF A SECTION (SERIES B) HAVING: BASE MIX; STANDARD FINISH; A SLUMP OF 1.2 INCHES; AND 40.8 PERCENT HONEYCOMB.

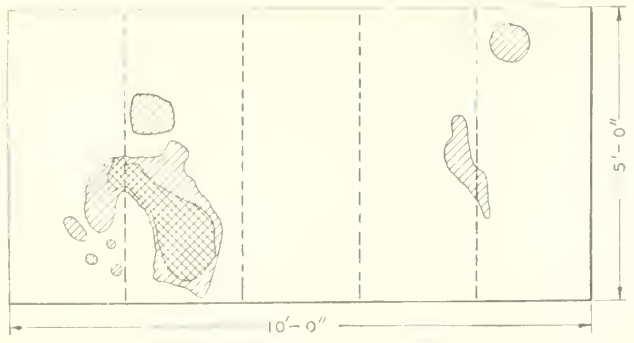


FIGURE 4.—DISTRIBUTION OF HONEYCOMB AT BOTTOM OF A SECTION (SERIES B) HAVING: BASE MIX; VIBRATED; A SLUMP OF 1.2 INCHES; AND 8.8 PERCENT HONEYCOMB.

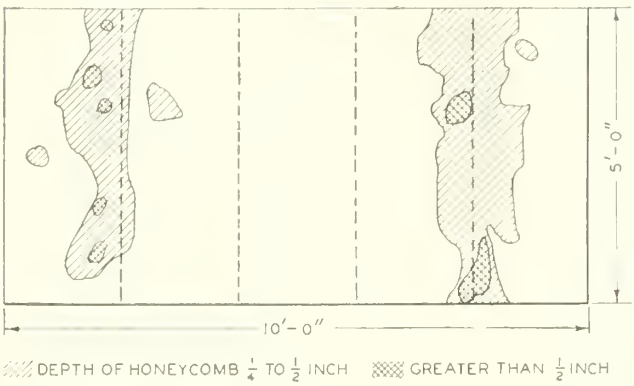


FIGURE 5.—DISTRIBUTION OF HONEYCOMB AT BOTTOM OF A SECTION (SERIES B) HAVING: ONE PART OF COARSE AGGREGATE ADDED TO BASE MIX; VIBRATED; A SLUMP OF 0.8 INCH; AND 20.4 PERCENT HONEYCOMB.

TABLE 6.—*Results of strength tests*

Section no	Modulus of rupture [1] at age of 9½ months		Crushing strength [2] at age of 14 months	
	Pavement slabs	Control beams	Pavement cores	Control cylinders
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
1-S [3]	707	579	7,340	4,900
2-S	695	650	6,400	5,390
3-V	682		7,820	6,320
4-V	643		6,910	5,380
5-V	696		7,270	5,540
6-V	642		7,430	5,340
7-S	685	571	7,820	5,480
8-S	688	628	8,050	6,150
9-V	711		8,450	6,130
10-V	673		6,880	5,280
11-V	[4] 674		6,620	5,180
12-V	[4] 599		6,920	5,090
13-S	663	637	7,360	5,900
14-S	655	630	7,930	6,030
15-V	594		7,540	6,570
16-V	647		7,840	6,370
17-V	[4] 661		8,080	5,420
18-V	575		6,800	5,990
19-S	656	591	6,150	5,760
20-S	637	639	8,360	5,800
21-V	681		8,080	6,000
22-V	695		6,920	5,490
23-V	692		7,590	5,590
24-V	656		7,100	5,960
25-S	686	681	8,410	5,890
26-S	602	669	8,030	6,390
27-V	750		8,300	6,150
28-V	712		8,180	6,440
29-V	715		7,880	5,680
30-V	712		7,450	5,980
31-S	881	741	7,760	6,470
32-S	886	834	8,700	6,600
33-V	998		8,880	6,510
34-V	884		8,060	6,190
35-V	868		8,400	6,360
36-V	852		8,350	6,090
37-S	836	720	7,170	6,640
38-S	[4] 876	807	7,540	6,570
39-V	989		8,130	6,790
40-V	[4] 952		8,520	7,080
41-V	848		8,140	6,780
42-V	798		7,580	6,720
43-S	896	720	7,400	6,280
44-S	[4] 918	755	7,830	6,850
45-V	898		7,580	6,650
46-V	887		7,600	6,620
47-V	927		8,260	6,980
48-V	931		8,320	6,860
49-V	[4] 973		7,800	6,930
50-S	894	689	7,060	6,000
51-S	[4] 913	685	7,480	6,260
52-V	916		7,880	6,500
53-V	973		8,060	6,070
54-V	921		8,280	6,740
55-V	885		7,200	6,290
56-V	[4] 877		8,020	6,510
57-S	[4] 672	659	6,760	5,480
58-S	752	657	7,460	5,870
59-V	770		8,330	6,330
60-V	765		7,170	6,670
61-V	691		7,660	6,140
62-V	698		7,890	6,170
63-V	733		7,920	6,490
64-S	726	622	7,180	5,420
65-S	745	658	7,180	5,730
66-V	763		7,630	6,190
67-V	746		7,900	5,870
68-V	754		7,620	6,370
69-V	756		6,980	6,250
70-V	702		7,400	6,280
71-S	674	592	6,680	5,230
72-S	720	617	7,720	5,960
73-V	698		7,920	5,870
74-V	658		7,120	5,560
75-V	667		6,560	5,480
76-V	668		6,520	5,390
77-S	703	600	6,380	5,070
78-S	698	627	7,330	5,860
79-V	712		7,190	6,130
80-V	662		6,500	4,920
81-V	693		6,860	5,170
82-V	675		6,160	4,930
83-S	665	588	7,140	5,090
84-S	734	603	6,730	5,910
85-V	701	[5] 616	7,440	5,840
86-V	672		6,040	4,930
87-V	609		6,420	5,030
88-V	639		6,420	5,350
89-S	[4] 794	704	6,620	5,780
90-S	817	712	6,600	6,260
91-V	812		6,600	6,190
92-V	767		7,030	5,910
93-V	814		6,450	5,910
94-V	781		6,340	5,650
95-S	679	732	5,780	5,830
96-S	714	730	6,260	6,650
97-V	768		6,210	6,320
98-V	732		6,460	5,640
99-V	722		6,680	5,370
100-V	716		6,700	5,900
101-S	554	602	5,660	5,830
102-S	558	626	6,160	5,960
103-V	628		6,440	6,130
104-V	609		5,800	5,580
105-V	596		5,850	5,310
106-V	[4] 583		6,270	5,100
107-S	662	572	6,560	5,490
108-S	686	661	6,990	6,140
109-V	715		7,330	5,980
110-V	667		6,500	5,310
111-V	668		6,410	5,250
112-V	598		5,860	4,750
113-S	640	533	7,050	5,510
114-S	675	652	6,240	5,830
115-V	732		7,150	5,890
116-V	715		7,320	5,520
117-V	691		7,490	5,680
118-V	719		7,140	6,190
119-V	742		6,780	6,640
120-S	671	611	6,680	5,610
121-S	738	619	7,350	5,930
122-V	745		7,640	6,090
123-V	698		7,280	6,100
124-V	741		7,880	6,600
125-V	726		7,680	5,590
126-V	737		7,520	6,450
127-S	662	579	6,260	5,160
128-S	659	633	6,840	5,950
129-V	719		7,240	5,920
130-V	656		6,300	5,360
131-V	688		6,620	5,210

[1] All slab values are the average of 5 tests unless otherwise noted. All beam values are the average of 4 tests unless otherwise noted.

[2] All core values are the average of 2 tests unless otherwise noted. All cylinder values are the average of 3 tests unless otherwise noted.

[3] Symbols used indicate: S-Standard finish. V-Vibrated finish.

[4] Average of less than 5 tests.

[5] Average of 3 tests.

TABLE 6.—*Results of strength tests*—Continued

Section no.	Modulus of rupture at age of 9½ months		Crushing strength at age of 14 months	
	Pavement slabs	Control beams	Pavement cores	Control cylinders
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
132–V	640		6,670	5,600
133–S	668	591	6,220	5,730
134–S	646	576	6,920	6,320
135–V	696		6,740	6,030
136–V	692		6,170	5,570
137–V	668		6,100	5,350
138–V	654		5,730	5,580
139–S	766	638	6,520	5,200
140–S	875	703	6,940	6,210
141–V	841		7,020	6,330
142–V	794		6,660	5,190
143–V	824		6,430	5,600
144–V	823		6,800	5,600
145–S	840	699	7,120	5,630
146–S	864	753	7,940	6,520
147–V	908		7,900	6,430
148–V	823		7,220	6,150
149–V	828		7,340	6,230
150–V	798		6,670	6,090
151–S	868	674	6,280	5,260
152–S	801	[5] 744	7,120	6,160
153–V	863	764	6,530	6,300
154–V	803	717	6,020	5,270
155–V	822	754	6,540	5,670
156–V	773	704	5,620	5,320
157–S	847	730	6,800	5,560
158–S	864	769	7,220	6,540
159–V	929	764	7,040	6,500
160–V	809	693	6,320	5,590
161–V	875	719	6,540	5,440
162–V	853	710	7,200	5,610
163–S	818	748	7,180	5,370
164–S	908	794	7,670	6,770
165–V	873	794	7,200	6,710
166–V	899	792	8,010	6,970
167–V	930	831	7,720	6,860
168–V	915	775	7,660	6,760
169–V	914	796	8,540	7,170
170–S	800	714	6,820	5,810
171–S	822	809	6,960	6,550
172–V	917	786	7,280	6,960
173–V	948	837	7,380	6,920
174–V	981	875	7,600	7,070
175–V	893	784	7,680	7,090
176–V	910	844	7,880	7,540
177–S	831	680	6,990	5,790
178–S	826	708	7,320	6,800
179–V	884	751	7,380	6,760
180–V	842	686	6,870	5,720
181–V	846	698	6,840	5,610
182–V	864	729	6,610	6,040
183–S	678	590	6,460	5,840
184–S	625	672	7,320	6,890
185–V	720	597	7,360	6,760
186–V	714	656	7,400	6,390
187–V	696		7,860	6,900
188–V	718		7,880	6,710
189–V	721		7,980	7,220
190–S	672	612	6,560	4,690
191–S	682	586	6,460	5,600
192–V	712	602	7,360	5,819
193–V	716	628	7,020	5,420
194–V	720	641	8,070	6,510
195–V	715	644	7,420	6,050
196–V	685	634	7,700	6,080
197–S	693	585	5,860	4,910
198–S	750	639	7,070	5,970
199–V	770	597	7,200	5,740
200–V	665	592	6,100	5,090
201–V	669	610	6,620	5,240
202–V	671	638	6,420	4,990
203–S	625	554	5,530	5,430
204–S	604	632	6,150	5,900
205–V	644	608	5,840	5,360
206–V	621	574	6,100	5,270
207–V	605	600	5,580	4,950
208–V	618	589	5,760	5,200
209–S	691	588	7,240	5,190
210–S	703	580	7,060	5,250
211–V	740	573	7,550	5,170
212–V	692	575	6,810	4,880
213–V	721	552	7,000	5,100
214–V	676	585	7,600	5,240
215–S	682	596	7,820	6,230
216–S	655	622	8,000	6,800
217–V	705	600	7,800	7,010
218–V	701	595	7,340	6,100
219–V	653	585	7,080	5,740
220–V	687	591	7,040	5,970
221–S	683	556	7,240	5,840
222–S	668	612	7,550	6,720
223–V	697	587	7,460	6,710
224–V	686	583	7,150	5,600
225–V	690	588	6,860	5,600
226–V	643	578	6,530	5,120
227–S	812	638	7,170	5,220
228–S	861	730	7,740	6,080
229–V	897	728	8,060	6,040
230–V	823	670	7,360	6,090
231–V	864	710	6,830	5,550
232–V	817	705	7,070	5,760
233–S	850	716	7,430	6,140
234–S	836	762	7,860	7,070
235–V	852	800	7,520	7,050
236–V	836	755	7,360	6,600
237–V	829	740	7,580	6,300
238–V	828	722	7,720	5,960
239–S	855	627	7,960	6,420
240–S	806	745	8,020	7,100
241–V	911	739	8,400+	7,470
242–V	852	725	8,130	6,340
243–V	793	719	7,780	6,260
244–V	787	681	7,460	6,010
245–S	739	726	5,800	6,420
246–V	772	728	7,130	6,550
247–S	779	688	6,430	6,140
248–V	843	712	7,140	5,990
249–V	820	734	6,820	6,060
250–V	767	[5] 777	6,780	6,380
251–S	793	673	6,720	6,500
252–S	757	729	7,560	7,400
253–V	778	758	7,140	7,420
254–V	770	754	7,520	6,650
255–S	843	694	6,860	6,520
256–S	837	736	8,080	7,240
257–V	829	769	7,240	6,970
258–V	760	790	7,420	6,810
259–S	776	680	6,580	5,920
260–S	836	708	8,320	6,940
261–V	889	688	6,960	6,520
262–V	853	694	7,180	6,200
263–V	859	673	7,150	6,190
264–V	845	730	7,100	6,470
265–S	851	712	7,400	6,710
266–S	878	639	7,100	6,440
267–V	846	681	7,360	6,810
268–V	847	652	6,760	6,480
269–V	829	695	6,850	6,180
270–V	783	678	7,140	6,350

[5] Average of 3 tests.

TABLE 7.—*Specific gravity and absorption of concrete slabs*

Section no.	Specific gravity of slab no.				Absorption of slab no.			
	1	3	5	Average	1	3	5	Average
					Percent	*Percent*	*Percent*	*Percent*
1	2.32	2.31	2.31	2.31	5.41	5.93	5.97	5.77
2	2.31	2.30	2.31	2.31	5.89	5.84	5.99	5.91
3	2.31	2.35	2.33	2.33	5.26	4.78	5.19	5.08
4	2.34	2.32	2.36	2.34	5.25	5.64	4.95	5.28
5	2.33	2.30	2.34	2.32	5.35	5.71	4.97	5.34
6	2.37	2.32	2.37	2.36	4.42	5.47	4.58	4.82
7	2.35	2.33	2.32	2.33	4.89	5.04	5.22	5.05
8	2.33	2.32	2.34	2.33	4.87	5.21	4.83	4.97
9	2.33	2.32	2.32	2.32	4.85	5.09	5.11	5.02
10	2.35	2.33	2.34	2.34	4.92	5.18	4.90	5.00
11	2.34	2.33	2.35	2.34	5.06	5.00	4.79	4.95
12	2.34	2.33	2.36	2.34	5.04	5.01	4.41	4.82
13	2.32	2.33	2.34	2.33	5.28	5.04	4.91	5.08
14	2.33	2.33	2.32	2.33	4.84	5.06	5.29	5.06
15	2.34	2.34	2.38	2.35	4.82	4.49	4.36	4.56
16	2.35	2.33	2.35	2.34	4.52	4.78	4.64	4.65
17	2.36	2.36	2.34	2.35	4.23	4.61	4.66	4.50
18	2.34	2.37	2.36	2.36	4.72	4.21	4.30	4.41
19	2.32	2.33	2.31	2.32	5.07	5.00	5.40	5.16
20	2.34	2.32	2.34	2.33	4.97	5.20	4.94	5.04
21	2.33	2.36	2.36	2.35	4.67	4.33	4.11	4.37
22	2.35	2.35	2.34	2.35	4.52	4.63	4.50	4.55
23	2.37	2.35	2.34	2.35	4.10	4.26	4.64	4.33
24	2.33	2.35	2.38	2.35	4.88	4.53	4.21	4.54
25	2.33	2.33	2.35	2.34	4.71	4.93	4.48	4.71
26	2.34	2.33	2.32	2.33	4.82	4.84	5.02	4.89
27	2.38	2.33	2.36	2.36	4.24	4.70	4.12	4.35
28	2.36	2.35	2.36	2.36	4.40	4.32	4.22	4.31
29	2.34	2.33	2.36	2.34	4.84	4.75	4.46	4.68
30	2.35	2.37	2.38	2.37	4.49	4.18	3.90	4.19
31	2.39	2.36	2.37	2.37	4.86	5.25	5.07	5.06
32	2.39	2.37	2.39	2.38	4.47	4.93	4.55	4.52
33	2.40	2.40	2.43	2.41	4.40	4.37	3.93	4.23
34	2.42	2.38	2.44	2.41	4.48	4.75	4.03	4.42
35	2.42	2.41	2.41	2.41	4.17	4.24	4.27	4.23
36	2.41	2.40	2.42	2.41	4.41	4.31	4.05	4.26
37	2.39	2.36	2.38	2.38	4.99	5.32	4.98	5.10
38	2.40	2.37	2.39	2.39	4.41	5.04	4.58	4.68
39	2.38	2.39	2.37	2.38	4.94	4.54	4.86	4.78
40	2.42	2.42	2.41	2.42	4.32	4.19	4.38	4.30
41	2.43	2.42	2.40	2.42	4.16	4.05	4.38	4.20
42	2.44	2.43	2.47	2.45	3.92	4.06	3.58	3.85

TABLE 7.—*Specific gravity and absorption of concrete slabs*—Con.

Section no.	Specific gravity of slab no.				Absorption of slab no.			
	1	3	5	Average	1	3	5	Average
					Percent	*Percent*	*Percent*	*Percent*
43	2.41	2.40	2.44	2.42	4.48	4.52	4.26	4.42
44	2.42	2.39	2.39	2.40	4.38	4.50	4.62	4.50
45	2.42	2.39	2.41	2.41	4.25	4.62	4.31	4.39
46	2.42	2.40	2.41	2.41	4.12	4.60	4.29	4.34
47	2.41	2.42	2.42	2.42	4.18	4.24	4.11	4.18
48	2.41	2.40	2.44	2.42	4.42	4.27	4.15	4.28
49	2.42	2.41	2.42	2.42	4.16	4.47	4.11	4.25
50	2.36	2.37	2.37	2.37	5.75	5.47	5.36	5.53
51	2.39	2.38	[1] 2.36	2.38	5.09	5.17	[1] 5.72	5.33
52	2.40	2.39	2.41	2.40	4.70	5.16	4.81	4.89
53	2.40	2.40	2.38	2.39	4.98	4.50	5.22	4.90
54	2.41	2.39	2.40	2.40	4.70	4.97	4.63	4.77
55	2.43	2.37	2.41	2.40	4.28	5.44	5.08	4.93
56	2.41	2.39	2.43	2.41	5.01	4.97	4.53	4.84
57	2.32	[1] 2.33	2.34	2.33	5.13	[1] 4.94	4.76	4.94
58	2.33	2.33	2.34	2.33	4.92	5.09	4.35	4.79
59	2.33	2.34	2.35	2.34	4.65	4.69	4.44	4.59
60	2.34	2.33	2.36	2.34	4.87	5.08	4.56	4.84
61	2.35	2.33	2.34	2.34	4.44	4.59	4.75	4.59
62	2.35	2.35	2.38	2.36	4.58	4.54	4.43	4.52
63	2.33	2.32	2.35	2.33	4.90	4.82	4.66	4.79
64	2.34	2.34	2.32	2.33	4.64	5.01	5.11	4.92
65	2.31	2.30	2.34	2.32	5.44	5.39	4.43	5.09
66	2.34	2.33	2.34	2.34	4.56	4.85	4.26	4.56
67	2.35	2.34	2.35	2.35	4.74	4.98	4.57	4.76
68	2.32	2.32	2.36	2.33	5.40	5.25	4.66	5.10
69	2.34	2.33	2.35	2.34	4.99	4.94	5.12	5.02
70	2.36	2.35	2.35	2.35	4.38	4.47	4.72	4.52
71	2.33	2.32	2.31	2.32	5.05	4.88	5.36	5.10
72	2.36	2.30	2.34	2.33	4.58	4.99	4.66	4.74
73	2.34	2.34	2.36	2.35	4.67	4.40	4.12	4.40
74	2.32	2.33	2.34	2.33	4.87	4.88	4.90	4.88
75	2.34	2.34	2.37	2.35	4.63	4.43	3.98	4.35
76	2.36	2.34	2.35	2.35	4.18	4.41	4.35	4.31
77	2.32	2.32	2.33	2.32	4.98	5.10	4.92	5.00
78	2.33	2.32	2.33	2.33	5.07	4.87	4.78	4.91
79	2.31	2.32	2.32	2.32	4.95	4.66	4.82	4.81
80	2.33	2.34	2.33	2.33	4.92	4.87	5.00	4.93
81	2.33	2.33	2.37	2.34	4.91	4.81	4.30	4.67
82	2.34	2.33	2.36	2.34	4.58	4.73	3.97	4.43
83	2.34	2.34	2.31	2.33	4.70	4.63	5.19	4.84
84	2.32	2.34	2.32	2.33	4.20	4.05	5.13	4.46
85	2.31	2.32	2.35	2.32	4.93	4.71	4.61	4.75
86	2.36	2.34	2.33	2.34	4.09	4.23	4.66	4.33
87	2.34	2.33	2.33	2.33	4.18	4.59	4.72	4.50
88	2.36	2.32	2.34	2.34	4.05	4.68	4.27	4.33
89	2.35	[2] 2.36	2.36	2.36	5.64	[2] 5.45	5.39	5.49
90	2.36	2.35	2.37	2.36	5.46	5.68	5.24	5.46
91	2.39	2.34	2.38	2.37	4.98	5.86	5.22	5.35
92	2.39	2.38	2.40	2.39	4.87	5.17	4.89	4.98
93	2.41	2.37	2.40	2.39	4.80	5.50	4.78	5.03
94	2.40	2.38	[1] 2.37	2.38	4.97	5.34	[1] 5.51	5.27
95	2.32	2.36	2.36	2.35	6.09	5.78	5.09	5.65
96	2.34	2.36	2.38	2.36	5.86	5.21	5.27	5.45
97	2.37	2.36	2.36	2.36	5.12	5.35	5.47	5.31
98	2.37	2.39	2.36	2.37	5.49	4.84	5.34	5.22
99	2.40	2.37	2.37	2.38	4.73	5.39	5.38	5.17
100	2.38	2.40	2.41	2.40	5.00	4.99	4.42	4.80
101	2.30	2.31	2.29	2.30	5.42	5.66	5.70	5.59
102	2.32	2.30	2.34	2.32	5.24	5.85	4.88	5.32
103	2.33	2.32	2.30	2.32	5.02	4.95	5.83	5.27
104	2.31	2.31	2.31	2.31	5.37	5.40	5.42	5.40
105	2.30	2.32	2.33	2.32	5.55	5.10	4.80	5.15
106	[2] 2.32	2.33	2.34	2.33	[2] 5.29	5.19	4.91	5.13
107	2.31	2.33	2.38	2.34	5.46	5.23	5.48	5.39
108	2.33	2.32	2.31	2.32	5.01	5.36	5.48	5.28
109	2.32	2.32	2.32	2.32	5.02	5.17	5.36	5.18
110	2.32	2.31	2.34	2.32	5.34	5.57	4.90	5.27
111	2.34	2.31	2.34	2.33	4.97	5.38	5.30	5.22
112	2.34	2.31	2.34	2.33	5.14	5.52	4.89	5.18
113	2.32	2.32	2.30	2.31	5.33	5.22	5.54	5.36
114	[2] 2.32	2.33	2.31	2.32	[2] 5.15	4.79	5.34	5.09
115	2.32	2.32	2.34	2.33	5.35	4.82	4.84	5.00
116	2.34	2.36	2.34	2.35	5.05	4.45	4.90	4.80
117	2.33	2.34	2.34	2.34	4.92	4.76	4.83	4.84
118	2.34	2.36	2.36	2.35	5.03	4.67	4.54	4.75
119	2.33	2.32	2.32	2.32	5.04	5.15	5.15	5.11
120	2.31	2.33	2.32	2.32	5.27	4.72	4.97	4.99
121	2.34	2.31	2.31	2.32	4.72	5.21	4.97	4.97
122	2.34	2.34	2.32	2.33	4.49	4.45	4.86	4.60
123	2.34	2.32	2.34	2.33	4.54	4.77	4.78	4.70
124	2.38	2.35	2.34	2.36	4.07	4.39	4.30	4.25
125	2.33	2.34	2.34	2.34	4.62	4.72	4.40	4.58
126	2.33	2.34	2.35	2.34	4.58	4.28	4.10	4.32
127	2.31	2.30	2.32	2.31	5.35	5.52	4.94	5.27
128	2.33	2.32	2.31	2.32	4.77	4.89	5.25	4.97
129	2.34	2.33	2.33	2.33	4.55	4.72	4.79	4.69
130	2.33	2.34	2.33	2.33	4.74	4.73	4.86	4.78
131	2.35	2.35	2.32	2.34	4.62	4.61	5.08	4.77
132	2.35	2.34	2.36	2.35	4.62	4.57	4.33	4.51
133	2.27	------	2.30	2.28	5.97	------	5.65	5.81
134	2.34	2.32	2.30	2.32	4.74	4.74	5.30	4.93
135	2.33	2.33	2.33	2.33	4.87	4.83	4.50	4.73
136	2.32	2.31	2.33	2.32	4.85	4.86	4.87	4.86
137	2.34	2.31	2.32	2.32	4.66	5.09	4.98	4.91
138	2.31	2.33	2.32	2.32	5.07	4.59	5.26	4.97
139	2.35	2.35	2.34	2.35	5.74	5.85	5.91	5.83
140	2.39	2.37	2.34	2.37	4.75	4.99	5.65	5.13
141	2.37	2.39	2.38	2.38	4.90	4.55	5.05	4.83

TABLE 7.—*Specific gravity and absorption of concrete slabs*—Con.

Section no.	Specific gravity of slab no.				Absorption of slab no.			
	1	3	5	Average	1	3	5	Average
					Percent	*Percent*	*Percent*	*Percent*
142	2.38	2.39	2.38	2.38	5.02	4.99	5.15	5.05
143	2.40	2.42	2.41	2.41	4.84	4.55	4.43	4.61
144	2.39	2.38	2.40	2.39	4.94	5.19	4.93	5.02
145	2.36	2.36	2.36	2.36	5.34	5.37	5.43	5.38
146	2.36	2.37	2.37	2.37	5.22	5.17	5.10	5.16
147	2.38	2.36	2.37	2.37	5.37	5.66	5.12	5.38
148	2.37	2.38	2.38	2.38	5.23	5.30	5.52	5.35
149	2.41	2.36	2.36	2.38	5.04	5.38	5.76	5.39
150	2.42	2.37	2.40	2.40	4.49	5.35	5.02	4.95
151	2.34	2.38	2.37	2.36	5.79	5.14	5.29	5.41
152	2.35	2.36	2.36	2.36	5.23	5.26	5.27	5.25
153	2.36	2.36	2.39	2.37	4.93	5.18	4.64	4.92
154	2.39	2.35	2.40	2.38	4.94	5.62	4.79	5.12
155	2.40	2.41	2.39	2.40	4.72	5.13	4.98	4.94
156	2.38	2.35	2.38	2.37	5.16	5.55	5.01	5.24
157	2.35	2.35	2.35	2.35	5.42	5.25	5.31	5.33
158	2.38	2.36	2.37	2.37	4.75	5.57	4.73	5.02
159	2.39	2.36	2.37	2.37	4.75	5.01	4.83	4.86
160	2.37	2.36	2.38	2.37	5.13	5.27	4.83	5.08
161	2.36	2.37	2.39	2.37	5.33	5.15	4.90	5.13
162	2.40	2.37	2.38	2.38	4.60	5.15	4.96	4.90
163	2.35	2.38	2.35	2.36	5.39	4.62	5.65	5.22
164	2.38	2.38	2.36	2.37	4.87	5.11	5.20	5.06
165	2.37	2.38	2.39	2.38	5.14	4.85	4.71	4.90
166	2.41	2.38	2.38	2.39	4.62	5.08	5.27	4.99
167	2.37	2.38	2.39	2.38	5.25	4.75	4.66	4.89
168	2.41	2.38	2.41	2.40	4.72	5.05	4.63	4.80
169	2.40	2.39	2.40	2.40	4.53	4.75	4.88	4.72
170	2.34	2.37	2.36	2.36	5.81	5.23	5.34	5.46
171	2.37	2.37	2.37	2.37	5.31	5.26	5.05	5.21
172	2.38	2.36	2.40	2.38	5.10	5.43	4.90	5.14
173	2.41	2.38	2.43	2.41	4.58	4.89	4.25	4.57
174	2.38	2.42	2.38	2.39	4.98	4.20	5.10	4.76
175	2.40	2.39	2.42	2.40	4.78	4.95	4.48	4.74
176	2.40	2.40	2.40	2.40	4.62	4.77	4.35	4.58
177	2.36	2.37	2.35	2.36	5.35	5.08	5.54	5.32
178	2.40	2.35	[1] 2.38	2.38	4.86	5.34	[1] 5.05	5.08
179	2.39	2.38	2.38	2.38	4.79	4.98	4.88	4.88
180	2.39	2.39	2.40	2.39	5.00	4.87	5.08	4.98
181	2.41	2.38	2.42	2.40	4.77	5.21	4.55	4.84
182	2.42	2.42	2.42	2.42	4.64	4.52	4.41	4.52
183	2.31	2.32	2.32	2.32	5.45	5.15	5.06	5.22
184	[3] 2.33	2.33	2.32	2.33	[2] 4.93	4.89	5.16	4.99
185	2.34	2.33	2.34	2.34	4.43	4.89	4.74	4.69
186	2.35	2.32	2.35	2.34	4.49	4.97	4.65	4.70
187	2.31	2.31	2.34	2.32	4.94	5.18	4.68	4.93
188	2.36	2.34	2.35	2.35	4.51	4.67	4.30	4.49
189	2.37	2.34	2.34	2.35	4.20	4.55	4.57	4.44
190	2.28	2.32	2.28	2.29	5.96	5.01	5.84	5.60
191	2.31	2.32	2.32	2.32	5.26	5.35	5.29	5.30
192	2.31	2.30	2.32	2.31	5.06	5.21	5.04	5.10
193	2.33	2.31	2.33	2.32	4.80	5.24	4.96	5.00
194	2.33	2.32	2.32	2.32	4.95	5.18	4.95	5.03
195	2.34	2.33	2.33	2.33	4.82	5.09	4.90	4.94
196	2.34	2.34	2.33	2.34	4.87	4.76	4.85	4.83
197	2.29	2.32	2.30	2.30	5.87	5.48	5.56	5.64
198	2.30	2.30	2.30	2.30	5.45	5.60	5.59	5.55
199	2.28	2.30	2.31	2.30	5.71	5.48	5.60	5.60
200	2.31	2.31	2.33	2.32	5.43	5.56	5.11	5.37
201	2.32	2.34	2.33	2.33	5.31	5.13	5.31	5.25
202	2.34	2.32	2.34	2.33	4.63	5.23	4.77	4.88
203	2.29	2.32	2.27	2.29	5.63	5.40	6.36	5.80
204	[2] 2.30	2.30	2.29	2.30	[2] 5.68	5.69	5.78	5.72
205	2.30	2.28	2.29	2.29	5.39	6.10	6.08	5.86
206	2.32	2.31	2.30	2.31	5.33	5.37	5.48	5.39
207	2.32	2.31	2.31	2.31	5.38	5.73	5.19	5.43
208	2.33	2.30	2.32	2.32	5.28	5.83	5.25	5.45
209	2.29	2.31	2.29	2.30	5.88	4.82	5.41	5.37
210	2.31	2.33	2.29	2.31	5.27	4.26	5.35	4.96
211	2.32	2.31	2.32	2.32	5.01	5.53	5.27	5.27
212	2.33	2.30	2.33	2.32	4.74	5.69	4.50	4.98
213	2.34	2.32	2.33	2.33	4.91	5.30	4.55	4.92
214	2.36	2.33	2.36	2.35	3.97	5.04	4.67	4.56
215	2.30	2.32	2.30	2.31	5.95	4.85	5.25	5.35
216	[2] 2.31	2.30	2.32	2.31	[2] 5.42	4.84	5.28	5.18
217	2.33	2.31	2.34	2.33	4.49	5.12	4.28	4.63
218	2.32	2.31	2.30	2.31	5.13	5.39	5.55	5.36
219	2.34	2.32	2.33	2.33	4.33	5.15	4.55	4.68
220	2.31	2.33	2.32	2.32	4.76	5.14	4.58	4.83
221	2.31	2.29	2.30	2.30	5.56	5.90	5.40	5.62
222	2.32	2.30	2.30	2.31	5.03	5.70	5.65	5.46
223	2.33	2.32	2.32	2.32	5.05	5.14	5.22	5.14
224	2.33	2.32	2.33	2.33	5.34	5.18	5.22	5.25
225	2.33	2.31	2.32	2.32	4.93	4.80	5.30	5.01
226	2.33	2.32	2.34	2.33	5.02	5.36	4.66	5.01
227	2.36	2.37	2.35	2.36	5.58	5.58	6.01	5.72
228	2.37	2.37	2.37	2.37	5.12	5.19	5.17	5.16
229	2.39	2.37	2.37	2.38	5.00	5.41	5.38	5.26
230	2.38	2.38	2.38	2.38	5.16	5.08	5.28	5.17
231	2.37	2.39	2.37	2.38	5.51	5.17	5.44	5.37
232	2.39	2.36	2.41	2.39	5.11	5.85	4.94	5.30
233	2.36	2.37	2.37	2.37	5.68	5.55	5.22	5.48
234	2.37	2.37	2.37	2.37	5.48	4.98	5.42	5.29
235	2.35	2.35	2.37	2.36	5.91	5.83	5.18	5.64
236	2.40	2.36	2.40	2.39	5.22	5.72	4.87	5.27
237	2.39	2.36	2.41	2.39	5.27	5.67	4.89	5.28

[1] Value for slab no. 4 used.
[2] Value for slab no. 2 used.

TABLE 7.—*Specific gravity and absorption of concrete slabs*—Con.

Section no.	Specific gravity of slab no.				Absorption of slab no.			
	1	3	5	Average	1	3	5	Average
					Percent	*Percent*	*Percent*	*Percent*
238	2.39	2.37	2.39	2.38	5.11	5.70	5.26	5.36
239	2.37	2.36	2.34	2.36	5.36	5.66	5.89	5.64
240	2.36	2.37	2.37	2.37	5.63	5.60	5.15	5.46
241	2.39	2.38	2.35	2.37	5.02	5.35	5.84	5.40
242	2.39	2.37	2.39	2.38	5.26	5.65	5.07	5.33
243	2.36	2.39	2.37	2.37	5.33	4.94	5.29	5.19
244	2.43	2.37	2.39	2.40	4.29	5.45	5.12	4.95
245	2.34	2.35	2.33	2.34	6.01	5.98	6.23	6.07
246	2.32	2.34	2.35	2.34	6.11	6.00	5.75	5.95
247	2.34	2.35	2.33	2.34	6.01	5.75	6.16	5.97
248	2.37	2.37	2.36	2.37	5.95	5.38	5.69	5.67
249	2.36	2.36	2.34	2.35	5.73	5.73	6.28	5.91
250	2.37	2.36	2.37	2.37	5.29	5.52	5.30	5.37
251	2.36	2.34	2.37	2.36	5.29	5.79	5.10	5.39
252	2.34	2.34	2.40	2.36	5.35	5.43	4.52	5.10
253	2.34	2.37	2.38	2.36	5.68	4.94	5.01	5.21
254	2.41	[1] 2.38	2.39	2.39	4.85	4.69	[1] 5.03	4.86
255	2.35	2.35	2.35	2.35	5.67	5.66	5.49	5.61
256	2.37	2.34	[1] 2.37	2.36	5.24	5.79	[1] 5.35	5.46
257	2.36	2.36	[1] 2.37	2.36	5.33	5.35	[1] 5.26	5.31
258	[2] 2.36	2.37	[1] 2.40	2.38	[2] 5.08	5.32	[1] 4.83	5.08
259	2.36	2.36	2.35	2.36	5.72	5.62	5.87	5.74
260	2.35	2.37	2.34	2.35	5.72	5.28	5.81	5.60
261	2.36	2.36	2.38	2.37	5.66	5.14	5.14	5.31
262	2.38	2.39	2.37	2.38	5.12	4.83	5.51	5.15
263	2.41	2.38	2.38	2.39	4.35	5.14	5.20	4.90
264	2.43	2.38	2.40	2.40	4.29	5.49	5.13	4.97
265	2.35	2.37	[1] 2.36	2.36	5.50	5.42	[1] 5.33	5.42
266	2.38	2.36	2.36	2.37	5.42	5.70	5.53	5.55
267	2.39	2.35	2.37	2.37	5.00	5.59	5.60	5.40
268	2.39	2.36	2.37	2.37	4.98	5.64	5.29	5.30
269	2.36	2.37	2.41	2.38	5.76	5.44	4.83	5.34
270	2.40	2.37	2.42	2.40	5.18	5.63	4.68	5.16

[1] Value for slab no. 4 used.
[2] Value for slab no. 2 used.

It will be evident from the foregoing that the major comparisons regarding the efficiency of surface vibrating equipment of the types now generally available are furnished by the tests in series B (see table 2). These results will therefore be presented first and the detailed discussion will be followed by a brief discussion of the results for series A and C.

The general effect of vibration on the strength and uniformity of concrete, as revealed by results obtained with machines A and B using both crushed stone and gravel, is shown graphically in figure 6. The average results of tests on 26 sets of 6 or 7 sections each, involving the standard operation of the vibrators, are given in this figure. Standard operation for machine A is defined as two passes of the machine, with both screeds vibrating. For machine B, standard operation is defined as one pass forward and back with pan vibrating, and a second pass made without vibration. The sections selected for study in the various figures can be identified by reference to table 2.

REDUCING SLUMP TO 1 INCH INCREASED HONEYCOMB IN NON-VIBRATED CONCRETE

In figure 6 the upper portion of each of the four blocks shows, for each variation in mix, (1) the relative flexural strength of the test slabs; and (2) the relative crushing strengths of cores drilled from test slabs expressed, in each case, as percentages of the strength of 2½-inch slump concrete finished without vibration. The lower portion of each block shows the corresponding data on uniformity indicated by: (1) The average percentage of variation in flexural strength of the five beams taken from each test slab; and (2) the average percentage of honeycomb in the slabs. Reading from left to right, the data shown in the four blocks indicate (1) the effect of increasing the amount of coarse aggregate in the mix while maintaining the water-cement ratio constant; (2) the effect of the same variation in mix for slabs 10 inches thick; (3) the effect of increasing the amount of both fine and coarse aggregate; and (4) the effect of changing the ratio of fine to coarse aggregate while maintaining the cement content constant.

In addition, there is also shown in each block (1) the relative strength and uniformity of standard-finished concrete having the same proportions as the base mix but with the water content reduced to give about a 1-inch slump; and (2) corresponding data for vibrated concrete of the same mix, consistency, and water content.

It is evident from the preceding discussion that by means of figure 6 a number of comparisons can be made, including:

1. The effect of changing the slump from about 2½ inches to 1 inch by decreasing the water-cement ratio with no change in the method of finishing, that is, without vibration (second panel in each block).

2. The effect of vibrating this 1-inch slump concrete (third panel in each block).

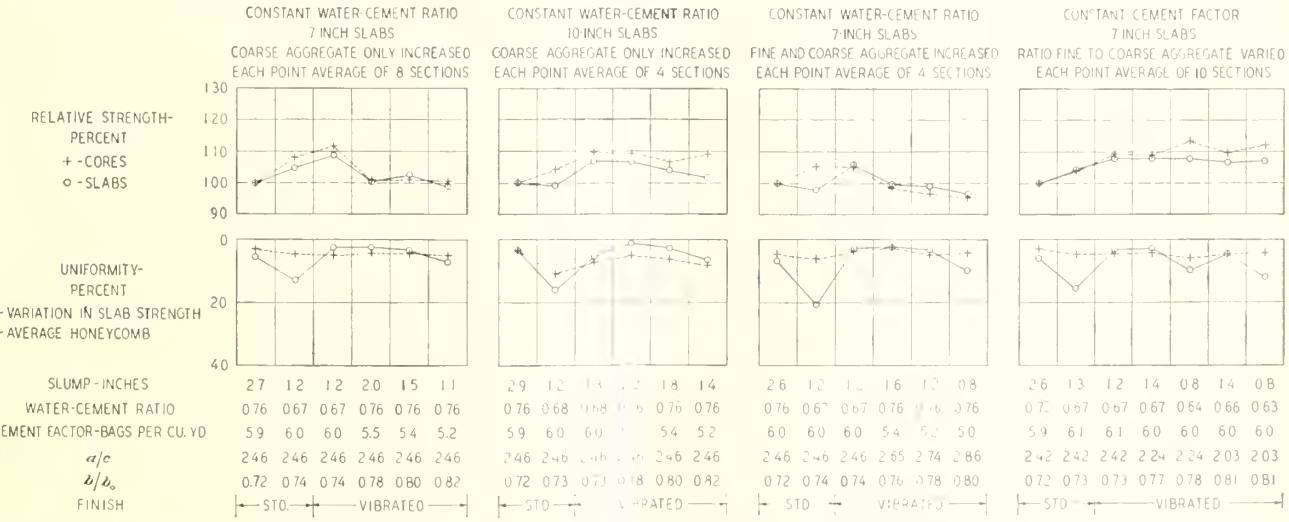
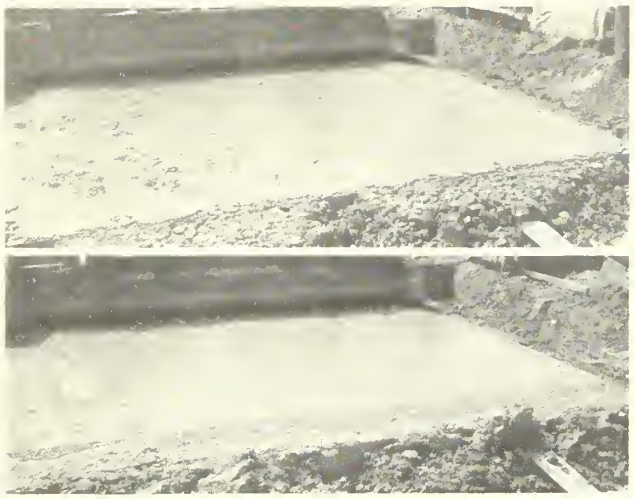


FIGURE 6.—EFFECT OF VIBRATION ON STRENGTH AND UNIFORMITY OF CONCRETE FINISHED BY STANDARD OPERATION OF VIBRATORS, SERIES B. EACH POINT IS THE AVERAGE FOR SECTIONS CONTAINING CRUSHED STONE AND GRAVEL FINISHED WITH MACHINES A AND B.



APPEARANCE OF 1-INCH SLUMP, BASE MIX CONCRETE AFTER FIRST PASS OF FINISHER. UPPER, NONVIBRATED SECTION; LOWER, VIBRATED SECTION

3. The effect of vibrating mixes harsher than the base mix, but having about the same water-cement ratio (fourth, fifth, and sixth panels in blocks 1, 2, and 3).

4. The effect of vibrating mixes harsher than the base mix, but having about the same cement factor (fourth, fifth, sixth, and seventh panels in block 4).

In studying this and subsequent charts, it should be recalled that in virtually all cases five slabs and three cores from each section were tested. Therefore almost every point in the diagrams represents a number of individual test values equal to five times the number of sections in the case of flexure, and three times the number of sections in the case of the cores. Results of 824 flexure tests and 498 compression tests are thus represented in figure 6.

In discussing figure 6, the several comparisons already noted will be considered in the order indicated.

The effect of reducing the slump to 1 inch without vibration is considered first.

In two of the four groups the average flexural strength was increased approximately 5 percent by reducing the slump to about 1 inch with a corresponding decrease in water-cement ratio. In the other two groups, one of which represents tests on the 10-inch sections, a slight decrease in flexural strength is noted. A weighted average value for the 22 sections of 7-inch concrete indicates an average increase in flexural strength of about 3 percent.

The average increase in flexural strength of the corresponding 22 sets of control beams was approximately 7 percent, a value that may be said to represent the normal increase in strength of the concrete when tested in the usual manner, that is, in the form of 7 by 7-inch beams molded in accordance with standard laboratory procedure.

The smaller increase in strength shown by the slabs as compared with the beams probably results from the lack of workability of the 1-inch slump concrete, as placed in the pavement. This has been noted in previous reports.

The lack of workability of the 1-inch slump concrete is also revealed by the high percentage of honeycomb found in the slabs. (See second panels in lower portion of each block and compare also figures 2 and 3.) In every case the tendency to honeycomb was markedly increased by decreasing the slump to 1 inch without vibrating the concrete. This same trend was noted in the earlier work and furnished the justification for the Bureau's requirement that 2-inch slump concrete be used in pavement construction.

Decreasing the slump to 1 inch slightly increased the average variation in slab strength in all three groups of 7-inch slabs and substantially increased the average variation in the 10-inch slabs.

In the matter of crushing strength of the cores, a weighted average for the 22 sections 7 inches thick shows an increase of about 6 percent as the result of reducing the water-cement ratio by 0.09. The average increase in cylinder strength for the same 22 sections was about 14 percent, which is about the amount that would theoretically be expected from a change in the water content of this magnitude.

The reason why the cores did not show a corresponding increase in strength is not definitely known. All were drilled from nonhoneycombed areas and it was expected that they would reflect the effect of changing the water content to about the same extent as the cylinders. However, attention is called to the fact that the cylinders were rodded in accordance with standard laboratory practice and that the standard-finished slabs made of 1-inch slump concrete were probably not adequately consolidated.

EFFECT OF SURFACE VIBRATION EXTENDED ENTIRELY THROUGH SLABS

The effect of vibrating 1-inch slump concrete of the same proportions as the base mix is considered next.

These values are shown in the third panel of each block in figure 6. Increase in flexural strength of vibrated concrete as compared with the base mix ranged from about 7 to 9 percent, with a weighted average for the twenty-two 7-inch sections of about 8 percent, or 5 percent higher than the nonvibrated, 1-inch slump concrete. The 10-inch slabs showed about the same average increase in strength as compared to the base mix. However, in this group the strength of the 1-inch slump, nonvibrated concrete was slightly less than that of the base mix. It is probable that in the case of the 10-inch slabs the lack of workability of the 1-inch slump concrete resulted in less uniform concrete under the standard method of finishing than in the 7-inch slabs. This is indicated by the relatively high variations in slab strengths shown for the nonvibrated concrete having a 1-inch slump.

The uniformity of concrete in the three groups of 7-inch slabs as revealed by the percentage of honeycomb was improved somewhat by vibration. In each of these groups, the average amount of honeycomb was slightly less than that shown by the base mix. In the case of the 10-inch slabs it was slightly greater. However, in all cases the amount of honeycomb in the 1-inch slump, vibrated concrete was very much less than in the similar mix finished by the usual method. (Compare also figs. 3 and 4.)

The comparatively large amount of honeycomb found in the 1-inch slump, vibrated concrete in the group of 10-inch slabs may indicate that, for this depth of section, a slump somewhat greater than 1 inch would probably prove more satisfactory when vibration is to be used. This is indicated by the improvement in uniformity shown in the vibrated sections in which the

average slump was 2.2 inches (panel 4). The very substantial reduction in the tendency of the 1-inch slump concrete to honeycomb on the bottom when vibrated seems to indicate definitely that the effect of the surface vibration of the concrete extended entirely through the slabs. This has been a disputed point ever since vibration was introduced.

Except for the 10-inch slabs, the average variation in slab strength was about the same as found for the base mix. The 10-inch slabs showed considerably less uniform results for the drier mixes.

The crushing strengths of cores from the twenty-two 7-inch sections of 1-inch slump, vibrated concrete averaged about 9 percent higher than for the base mix and about 3 percent higher than for the 1-inch slump, nonvibrated concrete. The crushing strengths of cores from the 10-inch sections showed about the same relative improvement. The increase in strength over the 1-inch slump, nonvibrated concrete was probably caused by vibration. This is indicated by the fact that the control cylinders, which were not vibrated, had almost exactly the same average crushing strength as the control cylinders representing nonvibrated concrete of the same consistency.

Further evidence of the undesirability of placing 1-inch slump concrete by the usual methods of finishing is afforded by the following comparison. The average percentage of variation in slab strength shown in figure 6 indicates the average uniformity of the concrete within a given test section. Uniformity may also be studied by comparing the average variation from the average flexural strength for the 22 sections of 7-inch concrete containing the base mix with similar variations for the corresponding 22 sections containing the 1-inch slump concrete, both nonvibrated and vibrated. Although these sections were laid on different days, they were laid in such a way as to eliminate the effect on these comparisons of variations in curing conditions.

The average percentage of variation in flexural strength for the 22 sections containing 2½-inch slump concrete was 2.9 percent, compared with 4.4 percent for the corresponding 22 sections containing the 1-inch slump concrete and 2.6 for the 22 vibrated sections of 1-inch slump concrete. These values are not shown in the figures or in table 4, but were obtained from data in table 6 for the sections represented in figure 6. It is interesting to note that the average day-to-day variation in flexural strength of the base mix (2.9 percent) was about the same as the average variation (3.2 percent) in the strength of the five slabs composing a given section. This would indicate that variations in curing and other conditions incidental to the average job do not cause greater variations in the strength of sections laid on different days than do factors such as depositing and spreading, which affect the uniformity of the concrete within a given section.

PRACTICAL APPLICATION OF FINDINGS

Depending upon the objective sought, existing specifications for pavement concrete may be modified to utilize vibration to advantage in either of the following ways:

1. By providing for adjustment of proportions to give a slump of approximately 1 inch with the same net water-cement ratio as is used in standard construction.

2. By providing for adjustment of proportions to give the same cement content as is used in standard construction but with the slump specified at 1 inch instead of 2½ inches.

In either case the specification should be worded so as to permit the engineer to vary the relative proportions of fine and coarse aggregate to produce the best results, depending upon the type and grading of the aggregates used and the type of finishing equipment employed.

VIBRATING ENABLED REDUCTION IN CEMENT CONTENT WITHOUT SACRIFICING STRENGTH

The effect of vibrating mixes harsher than the base mix is considered next. Mixes having a constant water-cement ratio and in which only the quantity of coarse aggregate was increased are discussed first. Data for the 7-inch sections are shown in the fourth, fifth, and sixth panels of the first block of figure 6.

In these sections the water-cement ratio used in the base mix was maintained constant, the slump being reduced by the addition of coarse aggregate to give average values of b/b_0 of 0.78, 0.80, and 0.82 as compared with 0.72 for the base mix. It will be observed that the average flexural and compressive strengths are both almost exactly the same as for nonvibrated base-mix concrete. The uniformity of the concrete as measured by the percentage of honeycomb was also about the same, although there was a tendency for the amount of honeycomb to increase as the mix became harsher. Uniformity as measured by variation in slab strength was about the same for all conditions in this group.

Corresponding values for the 10-inch slabs are shown in the second block of figure 6. In this case all of the harsher mixes when vibrated showed higher strengths both in flexure and compression than the base mix, although the flexural strength decreased as the percentage of coarse aggregate was increased. The uniformity of the harsher mixes as measured by honeycomb also compared favorably with the standard, although a decrease in uniformity was found as the amount of coarse aggregate was increased beyond a certain point. Uniformity as indicated by variations in slab strength was somewhat less than for the corresponding 7-inch sections.

It should be noted that in both the 7-inch and the 10-inch slabs the leanest and harshest mixes gave as high strengths as the base mix even though the cement factor averaged three-fourths of a sack per cubic yard less. The slump averaged slightly more than 1 inch and the water-cement ratio and sand-cement ratio were the same as for the base mix, that is 0.76 and 2.46, respectively.

Mixes having a constant water-cement ratio and in which both fine and coarse aggregate were increased are discussed next. Results are shown for the 7-inch slabs in the third block in figure 6. The results do not differ materially from those for the group in which only the amount of coarse aggregate was varied, although there tended to be a slightly greater reduction in

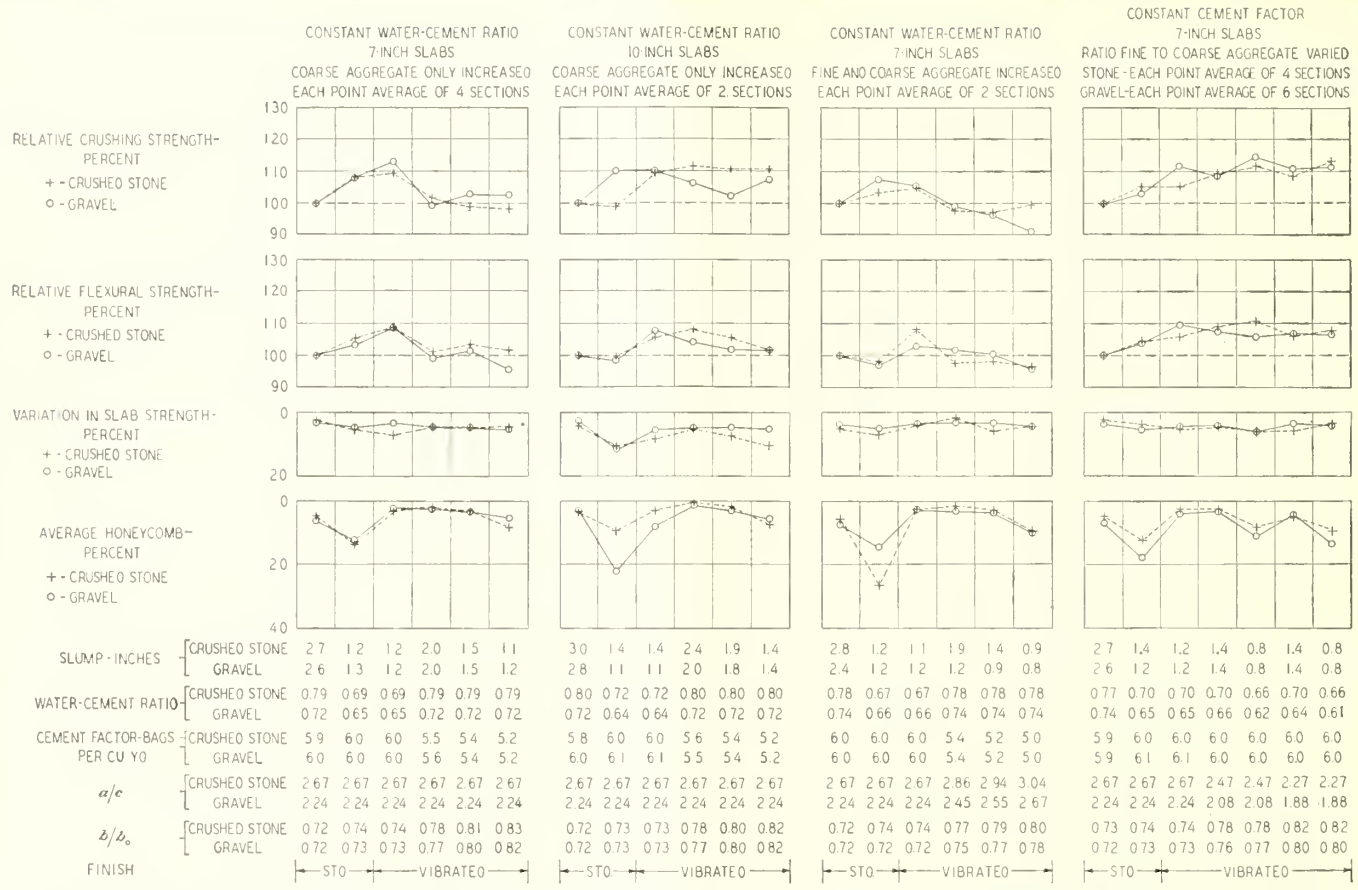


FIGURE 7.—EFFECT OF TYPE OF COARSE AGGREGATE ON STRENGTH AND UNIFORMITY OF CONCRETE FINISHED BY STANDARD OPERATION OF VIBRATORS, SERIES B. EACH POINT IS THE AVERAGE FOR SECTIONS FINISHED WITH MACHINES A AND B.

strength as the quantity of aggregate was increased. The percentage of honeycomb was substantially increased for the sections made of concrete having less than 1-inch slump, indicating that this slump is about the lowest that should be used with the type of equipment employed in these tests. This holds true even where the cement content was maintained constant, as will be noted by referring to the fifth and seventh panels in the fourth group, where the slump was also less than 1 inch.

It will be noted that the harshest mix in group 3 contained somewhat less cement and had less slump than the corresponding sections in groups 1 and 2. This resulted from the increased sand content. The results seem to indicate that for concrete vibrated with equipment of the type used in these tests the slump should not be less than 1 inch and that the cement factor should not be reduced by more than three-fourths sack per cubic yard. The results also show that nothing is gained either in strength or uniformity by increasing the sand content.

Mixes having a constant cement factor and in which the ratio of fine to coarse aggregate was varied are discussed next. It will be observed that the strengths of all the vibrated sections were greater than that of the base mix. This applies to both flexure and compression. The flexural strength remained virtually the same but the crushing strength increased in the two cases where the slump was reduced to 0.8 inch by lowering the water-cement ratios to 0.64 and 0.63, the lowest average values used in the test. It would appear from these data that the flexural strength of concrete is not affected appreciably by varying the ratio of fine to coarse aggregate within the limits used in these tests. However, the tendency to honeycomb is increased by drying the mix to less than 1-inch slump.

The comparatively high crushing strengths of the sections containing the 0.8-inch slump concrete is explained by the fact that the cores were drilled from nonhoneycombed areas. Under these conditions the strengths paralleled the reduction in water-cement ratio. These data illustrate the danger of drawing conclusions regarding the quality of paving concrete solely from the results of core tests.

VIBRATED CONCRETE HAD SLIGHTLY GREATER SPECIFIC GRAVITY AND LESS ABSORPTION

In figure 7 the data shown in figure 6 have been plotted to indicate the effect of the type of coarse aggregate on the results. Except for group 4, each point represents just half the number of sections shown in the corresponding panels of figure 6. Comparing relative crushing strengths first, it will be observed that, except for the 10-inch slabs, there was no consistent difference that may be attributed to the coarse aggregate. For the 10-inch slabs, three of the vibrated crushed-stone sections had higher relative strengths than the corresponding gravel sections. There likewise appeared to be no consistent difference in relative flexural strength of 7-inch sections. This does not bear out the conclusion drawn as the result of the work in 1931, where a considerably greater relative increase in flexural strength under vibration was indi-

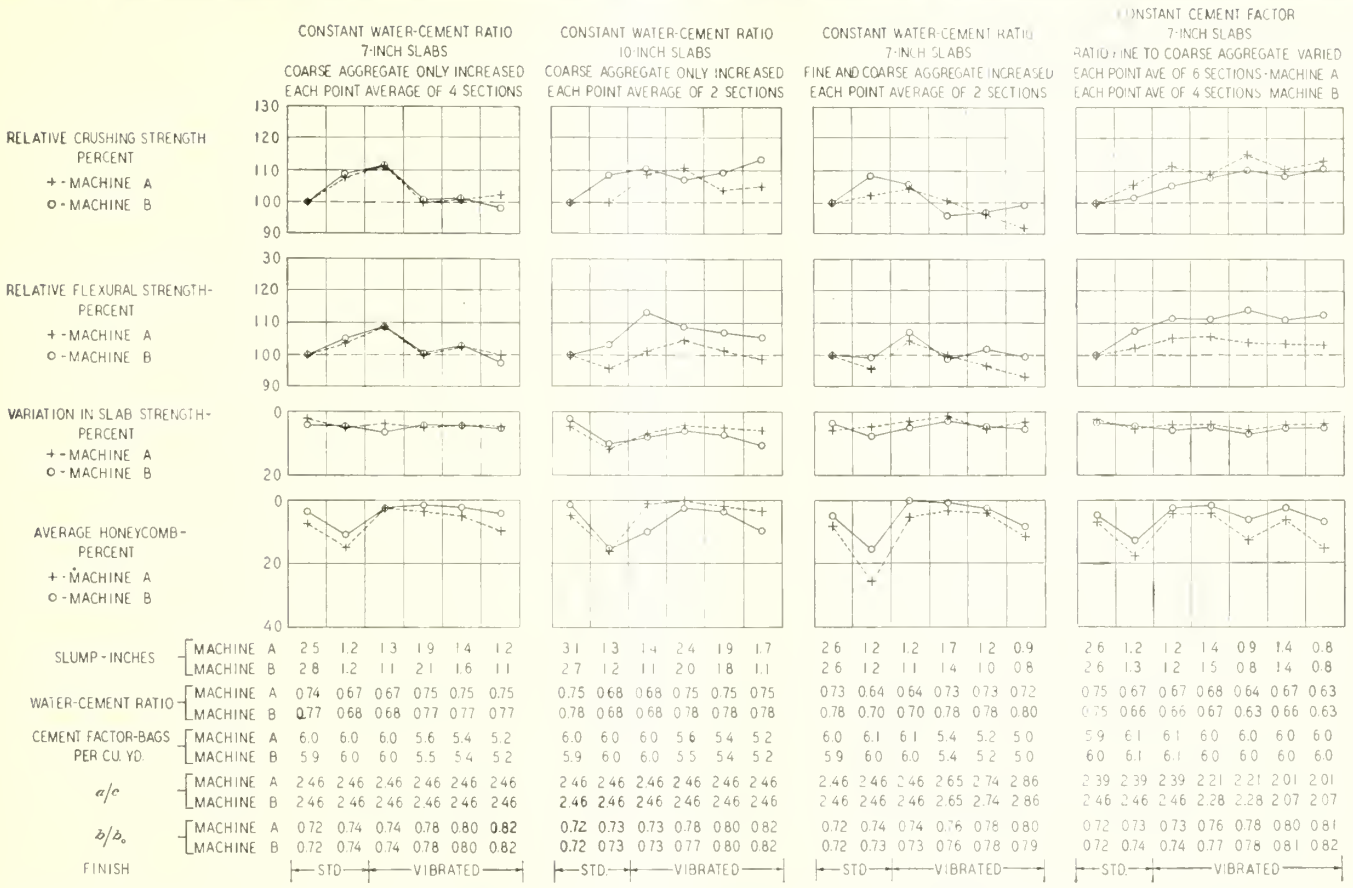


FIGURE 8.—EFFECT OF TYPE OF FINISHING MACHINE ON STRENGTH AND UNIFORMITY OF CONCRETE FINISHED BY STANDARD OPERATION OF VIBRATORS, SERIES B. EACH POINT IS THE AVERAGE FOR SECTIONS CONTAINING CRUSHED STONE AND GRAVEL.

cated when crushed stone was used as the coarse aggregate.

In the matter of uniformity there is also little to choose between the gravel and crushed-stone aggregates. The greatest difference was in the percentage of honeycomb in the group of 10-inch sections. Here, for some reason, the gravel concrete showed more honeycomb, especially in the sections having 1-inch slump, nonvibrated concrete. However, it should be noted that the average slump of the gravel concrete in these sections was somewhat less than that of the corresponding crushed-stone concrete (see group 2, panel 2, in fig. 7).

Results obtained using machines A and B are compared in figure 8. Considering only crushing strength, there appeared to be no consistent difference in the effectiveness of the two types of vibrating equipment except in group 4, which showed a slight advantage in favor of machine A. On the other hand, the flexural strength results indicated, in two of the four groups, a consistent advantage in favor of machine B. In group 1 there was virtually no difference, while in group 3 there was a slight trend in favor of machine B. On the whole, machine B seemed to give slightly better results as far as increase in flexural strength is concerned. From the standpoint of variation in slab strength (uniformity) the tendency is in the other direction.

There also seemed to be a slight tendency for less honeycomb in concrete finished by machine B, except in the 10-inch slabs, where the trend was reversed. Here again, the difference may result from the slightly lower average slump for the concrete finished with machine B.

In studying figure 8 it should be borne in mind that the values indicate, for each type of equipment, the relative strength obtained when the concrete was vibrated compared with the strength of the concrete finished by the same machine operating without vibration. Differences in quality resulting from variations in the design or operation of the two machines other than the vibrating equipment are not shown. Such comparisons can be made by comparing the average strengths obtained on the base concrete by the two machines. These comparisons, however, are of doubtful value because of the possible effect of variation in curing and other conditions resulting from the fact that the two machines were always used on different days.

The average values for bulk specific gravity and absorption of 6-inch concrete cores drilled from sections in series B are shown in figure 9. The same grouping is used as in figures 6 to 8, inclusive. These values reflect in a general way variations in density of the concrete, as revealed by tests on portions of the slabs free from honeycomb. They bear no relation necessarily to the average density of the slabs as a whole, or to the density of any portion in which honeycombed areas may be included. The differences shown are not of great magnitude in any case, although there seems to be a slight tendency for vibrated sections to have increased specific gravity and lower absorption. It will also be observed that, in general, density increases

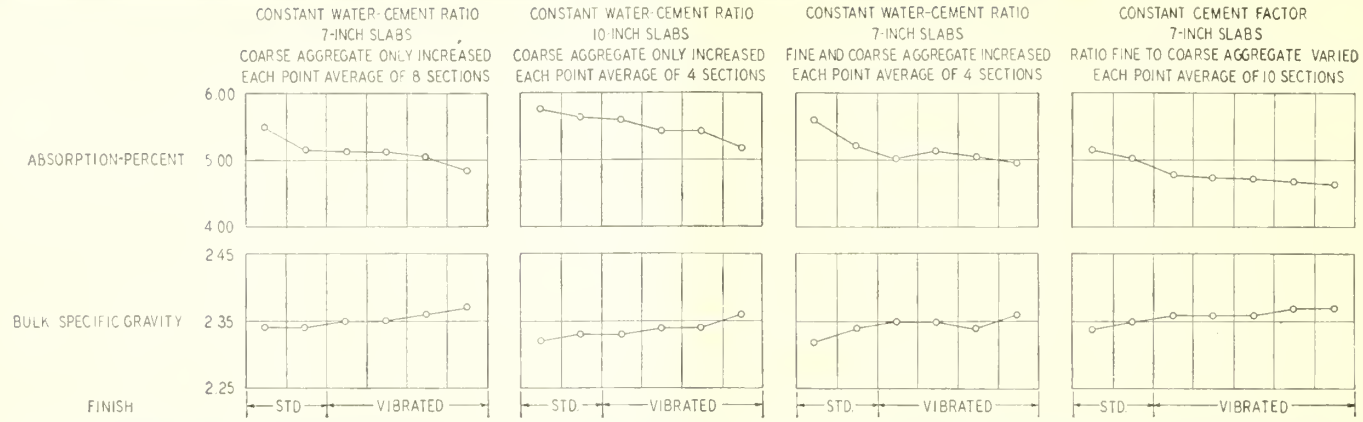
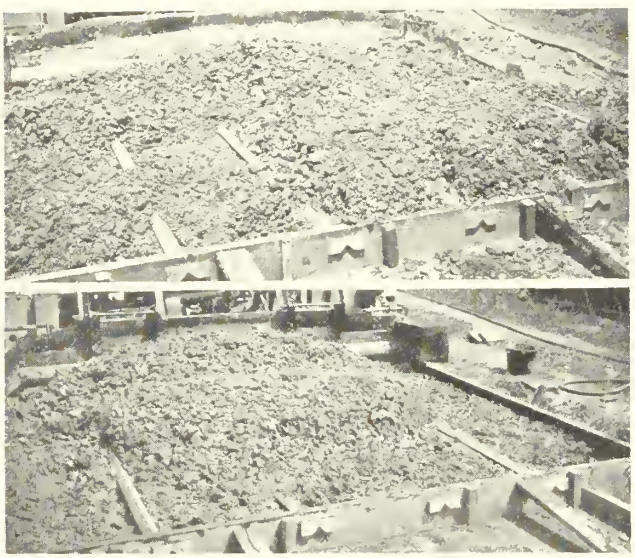


FIGURE 9.—GENERAL AVERAGE VALUES FOR ABSORPTION AND SPECIFIC GRAVITY OF CONCRETE FINISHED BY STANDARD OPERATION OF VIBRATORS, SERIES B. EACH POINT IS THE AVERAGE FOR SECTIONS CONTAINING CRUSHED STONE AND GRAVEL FINISHED WITH MACHINES A AND B. SECTIONS, READING FROM LEFT TO RIGHT, ARE THE SAME AS IN FIGURE 6.



APPEARANCE OF TEST SECTIONS AFTER SECOND BATCH HAS BEEN DEPOSITED BUT NOT SPREAD. UPPER, CONCRETE DEPOSITED BY MIXER DISCHARGE BUCKET BY METHOD USED IN SERIES A; LOWER, CONCRETE DEPOSITED BY METHOD USED IN SERIES B AND C.

as the proportion of coarse aggregate in the concrete increases.

As was indicated in the previous report, it is probable that the harsher concretes, when freshly mixed, contained more air voids than the base mix. The fact that the final densities of these harsh mixtures, as revealed by bulk specific gravity tests on the cores, tended to arrange themselves in about the same order as the theoretical densities (ratio of the sum of the absolute volume of solids to volume of concrete) indicates that the increased densities corresponding to the changes in composition were actually being obtained as the result of the method of compaction used.

WIDE VARIATION IN STRENGTH AND HONEYCOMBING ACROSS SECTION REMEDIED BY VIBRATING

It is evident that one of the benefits derived from the use of vibration in these tests was improvement in the uniformity of the 1-inch slump concrete, obtained by reducing the amount of honeycomb in the slabs. The general relation between honeycomb and slump is shown in figure 10. It will be observed that the tendency of the standard-finished concrete to honeycomb increased rapidly for a slump of about 1 inch, and that a slump of about 2½ inches was required to reduce the honeycomb to 5 percent. On the other hand, the same degree of uniformity was obtained with the vibrated concrete at about 1-inch slump.

Reference to figures 2 to 5, inclusive, which show the distribution of honeycomb on the bottom of sections, illustrates the extent to which honeycombing was concentrated in the outer slabs (numbers 1 and 5) of the test sections.

It will be recalled that in this series the concrete in the two outside slabs was distributed by hand shoveling from the mass as deposited by the mixer discharge bucket. This method of distributing the concrete along the forms and in the corners formed by joints, etc., is practiced to a certain extent on all paving jobs, although efforts are made on well-conducted jobs to eliminate as much of this hand work as possible. Figure 3 illustrates the excessive amount of honeycombing that is apt to form in concrete that is shoveled into place and then finished in the ordinary way. Apparently the surface vibrators were effective in reducing honeycombing because, as shown in figure 6, the average honeycomb in the sections containing the 1-inch slump, vibrated concrete was less even than in the 2½-inch slump, standard-finished concrete and very much less than in the 1-inch slump concrete finished in the usual way.

The effect of the location of the test slab in the section on the strength and amount of honeycomb in the concrete is shown in figure 11. The upper portion of the chart gives for each of the five slabs the average flexural strength of the three groups of twenty-two 7-inch sections shown in figure 6, in which the base proportions were used. The lower portion of figure 11 shows the corresponding percentages of honeycomb in the slabs. Each point is the average of 22 individual values. This figure is of interest particularly in showing how the nonuniform distribution in slabs nos. 1 and 5 in the 1-inch slump, standard-finished concrete affected the strength. In both outer slabs the average strength of the 1-inch slump, standard-finished concrete was less than the strength of the 2½-inch slump, standard-finished concrete. The effect of vibration on honeycomb is also brought out clearly. Note that the average percentage of honeycomb in the 22 no. 1 slabs was reduced from 26 to 3 percent, and in the 22 no. 5 slabs from 31 to 4 percent.

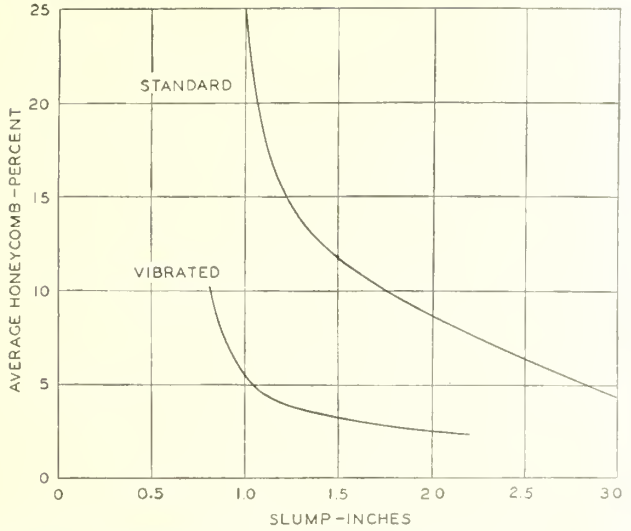


FIGURE 10.—RELATION BETWEEN HONEYCOMB AND SLUMP. SERIES B; 7-INCH SLABS; AVERAGE FOR STONE AND GRAVEL; MACHINES A AND B; STANDARD OPERATION OF VIBRATORS.

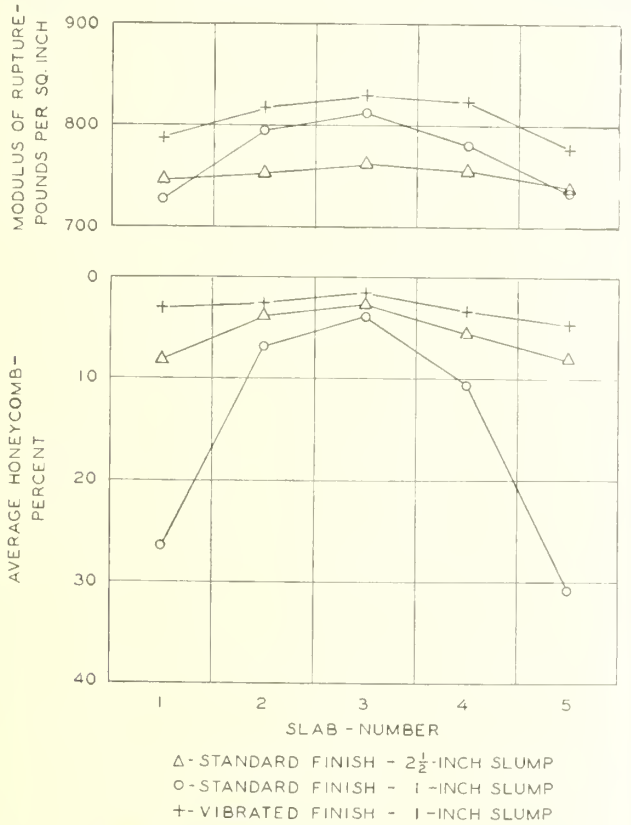


FIGURE 11.—RELATION BETWEEN SLAB STRENGTH AND HONEYCOMB. STANDARD OPERATION OF VIBRATORS; 7-INCH SLABS; SERIES B; AVERAGE FOR MACHINES A AND B; STONE AND GRAVEL.

Also, note that in the case of slab no. 3 the nonvibrated, 1-inch slump concrete showed an average strength about 50 pounds per square inch higher than the base mix, 2½-inch slump concrete. Slab no. 3, being directly under the bucket when dumped, is probably in the most favorable location as regards the effect of consistency upon uniformity. This is indicated by the fact that the average increase in strength of slab no. 3 resulting from the use of 1-inch slump concrete was about

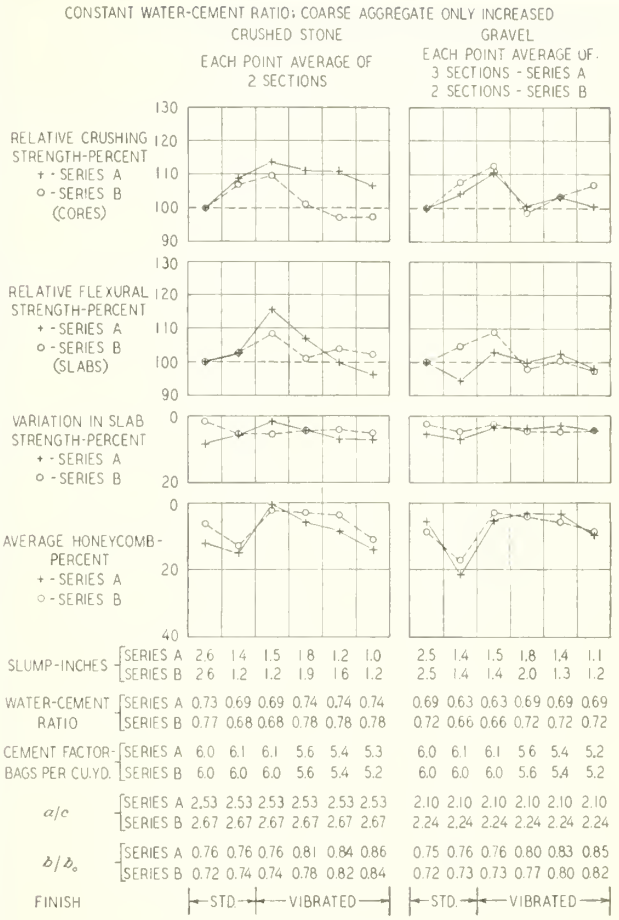

CONSTANT WATER-CEMENT RATIO; COARSE AGGREGATE ONLY INCREASED
CRUSHED STONE
EACH POINT AVERAGE OF 2 SECTIONS
GRAVEL
EACH POINT AVERAGE OF: 3 SECTIONS - SERIES A, 2 SECTIONS - SERIES B
RELATIVE CRUSHING STRENGTH-PERCENT
+ - SERIES A
○ - SERIES B
(CORES)
RELATIVE FLEXURAL STRENGTH-PERCENT
+ - SERIES A
○ - SERIES B
(SLABS)
VARIATION IN SLAB STRENGTH-PERCENT
+ - SERIES A
○ - SERIES B
AVERAGE HONEYCOMB - PERCENT
+ - SERIES A
○ - SERIES B
SLUMP-INCHES
SERIES A 2.6 1.4 1.5 1.8 1.2 1.0 2.5 1.4 1.5 1.8 1.4 1.1
SERIES B 2.6 1.2 1.2 1.9 1.6 1.2 2.5 1.4 1.4 2.0 1.3 1.2
WATER-CEMENT RATIO
SERIES A 0.73 0.69 0.69 0.74 0.74 0.74 0.69 0.63 0.63 0.69 0.69 0.69
SERIES B 0.77 0.68 0.68 0.78 0.78 0.78 0.72 0.66 0.66 0.72 0.72 0.72
CEMENT FACTOR - BAGS PER CU.YD.
SERIES A 6.0 6.1 6.1 5.6 5.4 5.3 6.0 6.1 6.1 5.6 5.4 5.2
SERIES B 6.0 6.0 6.0 5.6 5.4 5.2 6.0 6.0 6.0 5.6 5.4 5.2
a/c
SERIES A 2.53 2.53 2.53 2.53 2.53 2.53 2.10 2.10 2.10 2.10 2.10 2.10
SERIES B 2.67 2.67 2.67 2.67 2.67 2.67 2.24 2.24 2.24 2.24 2.24 2.24
b/b₀
SERIES A 0.76 0.76 0.76 0.81 0.84 0.86 0.75 0.76 0.76 0.80 0.83 0.85
SERIES B 0.72 0.74 0.74 0.78 0.82 0.84 0.72 0.73 0.73 0.77 0.80 0.82
FINISH
STD. VIBRATED
STD. VIBRATED

FIGURE 12.—COMPARATIVE SLAB STRENGTH AND UNIFORMITY FOR SERIES A AND B. STANDARD OPERATION OF MACHINE A; 7-INCH SLABS.

7 percent. This is almost exactly the same as the average increase in strength of the corresponding 22 sets of control beams and, as noted, probably represents the normal increase in flexural strength to be expected for these materials caused by a decrease in water content of approximately ¾ gallon per sack of cement.

EFFECTIVENESS OF THE VARIOUS TYPES OF VIBRATING EQUIPMENT COMPARED

As indicated earlier in the report, the first 42 sections, constituting series A, were constructed with a base mix that appeared somewhat undersanded at the time of construction. For this reason it was deemed advisable to repeat in series B that portion of series A involving the standard operation of machine A. This makes possible comparison of the results obtained with machine A in the two series. Data for comparable sections are shown in figure 12. They involve only groups of sections in which the proportions of the base mix were varied by increasing the amount of coarse aggregate.

The two series show, in general, the same trends insofar as the effect of vibration is concerned. The unusually high crushing strengths of the vibrated mixtures in series A, which contained increased quantities of crushed stone, and the relatively low flexural strength of the 1-inch slump, standard-finished gravel concrete, are the principal exceptions. It will be observed that, in general, a somewhat greater uni-

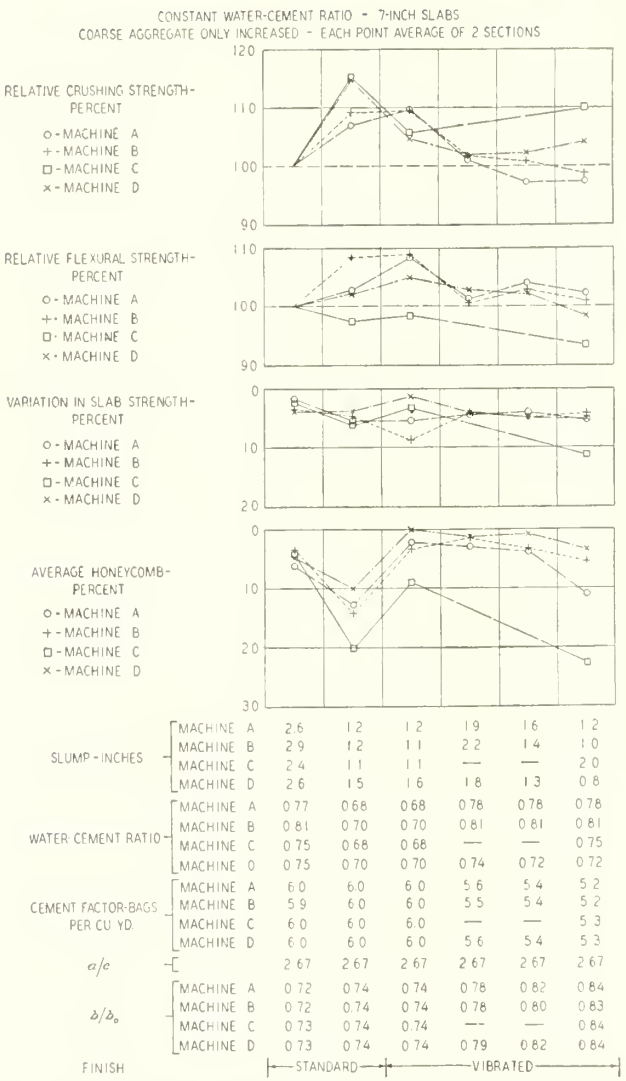


FIGURE 13.—EFFECT OF TYPE OF FINISHING MACHINE ON STRENGTH AND UNIFORMITY OF CONCRETE FINISHED BY STANDARD OPERATION OF VIBRATORS, AND HAVING CRUSHED STONE ONLY. DATA FOR MACHINES A AND B ARE FROM SERIES B; DATA FOR MACHINES C AND D ARE FROM SERIES C.

formity in strength was obtained in series B. This seems reasonable in view of the adjustment made in sand content. However, as shown in the third panel of each block, vibration was about as effective in reducing honeycomb in the base mix in series A as in series B. With increasing quantities of crushed stone the amount of honeycomb increased at a somewhat greater rate in series A. The high percentage of honeycomb, high average variation, and low strength of the harshest crushed-stone mix indicates that the maximum safe value of b/b_0 for vibrated crushed-stone concrete had been exceeded in series A.

In addition to the tests that have been discussed, a few sections were constructed on which the operation of the vibrating equipment was modified to a certain extent. For tests with machine A the modification consisted of operating the finishing machine with only the front screed vibrating. This method was used on four sets of sections—two in series A and two in series B. In the case of machine B the vibrating procedure was changed by vibrating during the forward movement of the machine only. Certain data from these tests have been grouped in table 8.

Referring first to the results obtained with only the front screed of machine A vibrating, and comparing the strength results obtained by vibrating the standard proportion, 1-inch slump concrete with the standard-finished base mix, a slight improvement was noted in two groups of sections only. However, the average percentage of honeycomb was reduced in three of the four groups. The results, in general, indicate that the modified operation was not as efficient as when both screeds were vibrating.

On five groups of sections the operation of machine B was modified so as to omit vibration during the backward movement of the machine. These results are also shown in table 8. In most cases they indicate a slight improvement in the vibrated concrete, but in general the improvement was not as great as was obtained when the vibrating pan was operated during both the forward and backward movement of the machine. Here also vibration seemed to have been generally effective in reducing the amount of honeycomb in the slabs.

TABLE 8.—*Results of special operation of machines A and B; concrete of base proportions only*

MACHINE A.—FRONT SCREED ONLY VIBRATING; 2 PASSES

Section and finish [1]	Consistency, slump	Average modulus of rupture (slabs)	Average crushing strength (cores)	Average honeycomb
	Inches	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*
7-S	3.0	685	7,820	5.9
8-S	1.3	688	8,050	5.0
9-V	1.2	711	8,450	.4
13-S	2.4	663	7,360	10.5
14-S	1.1	655	7,930	30.6
15-V	1.0	594	7,540	33.1
215-S	2.4	682	7,820	5.9
216-S	1.0	655	8,000	38.4
217-V	1.0	705	7,800	7.4
233-S	2.4	850	7,430	10.3
234-S	1.0	836	7,860	35.7
235-V	1.0	852	7,520	5.6
MACHINE B—PAN VIBRATING FORWARD ONLY				
71-S	2.6	674	6,680	8.2
72-S	1.1	720	7,720	13.4
73-V	1.2	698	7,920	4.4
77-S	2.5	703	6,380	5.1
78-S	1.2	698	7,330	3.3
79-V	1.0	712	7,190	3.3
83-S	3.0	665	7,140	2.3
84-S	1.4	734	6,730	4.0
85-V	1.4	701	7,440	4.5
89-S	2.4	794	6,620	0
90-S	1.4	817	6,600	4.5
91-V	1.2	812	6,600	0
151-S	2.7	868	6,280	3.8
152-S	1.1	801	7,120	16.5
153-V	1.2	863	6,530	4.4

[1] Sections 89-91, 151-153, and 233-235 contained crushed stone; balance of sections contained gravel.

The results obtained with crushed-stone concrete, using machines C and D, are shown in figure 13, together with corresponding data for machines A and B. Each point is the average of two sections. This figure compares the four types of surface vibrators that were investigated. Comparing first the relative strength of the virbated concrete of the base proportions (third panel), it will be observed that neither machine C nor machine D gave as good results as machines A and B. This applies to both flexure and compression. Machine D, however, compares favorably with the other type as regards the strength and uniformity of the harsher mixes. Machine C appears to be the least efficient insofar as flexural strength is concerned. Machine C appears to be less efficient

than the other types from the standpoint of eliminating honeycomb from the slabs. Machine D was quite effective in reducing honeycomb in the vibrated concrete slabs.

Referring to table 6, it will be noted that almost without exception the strengths of the control beams and control cylinders were lower than the strengths of the corresponding pavement slabs and pavement cores. This parallels previous observations made in connection with similar work. An analysis of test data for the control specimens reveals a number of interesting relationships, among which may be noted the variations in strength obtained on concrete of the same proportions and water content but placed on different days. For instance it will be recalled that the average variation in flexural strength of the twenty-two 7-inch, standard-finished sections in series B that contained 2½-inch slump concrete was 2.9 percent. The average variation for the corresponding 22 control beams was 4.4 percent, indicating that control specimens tend to have wider variations in strength than are to be found in pavement slabs.

The corresponding variations in values for crushing strength were 4.6 percent for the cores and 5.2 percent for the cylinders.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MARCH 31, 1937

STATE	APPORTIONMENT 1/	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,872,980	$ 51,600	$ 25,800	9.0	$ 1,374,281	$ 687,130	71.2	$ 339,290	$ 169,640	12.1	$ 6,990,410
Arizona	5,394,661	2,021,783	1,564,817	116.6	1,451,761	1,095,808	47.4	381,341	272,720	15.9	2,461,315
Arkansas	6,463,681				1,613,679	1,612,354	75.8	1,737,375	1,736,146	118.4	3,115,181
California	14,366,891	5,987,362	3,408,467	151.4	8,587,612	4,899,585	225.4	1,982,772	1,005,678	34.6	5,053,161
Colorado	6,911,198	3,974,044	2,113,986	160.1	3,415,669	1,913,613	129.2	784,462	438,256	33.3	2,445,343
Connecticut	2,388,339	771,100	385,412	14.2	705,928	350,556	8.7				1,652,370
Delaware	1,843,750	576,378	280,117	27.5	364,282	181,956	8.5	217,279	108,635	5.7	1,273,042
Florida	5,020,323	1,016,214	502,728	31.5	1,703,255	851,623	57.9	609,970	304,985	7.4	3,360,987
Georgia	9,569,722	1,488,755	703,017	102.9	2,443,984	1,221,977	129.7	932,680	466,340	46.2	7,178,388
Idaho	4,635,991	2,661,103	1,544,662	256.6	1,097,785	656,844	72.7	580,515	347,395	29.3	2,087,090
Illinois	15,564,720	8,342,942	4,155,155	134.5	5,681,179	2,785,823	153.0	4,228,678	2,111,477	110.4	6,512,264
Indiana	9,333,269	5,393,789	2,670,751	179.1	4,201,102	2,099,713	86.5	2,624,393	1,312,119	88.1	3,250,687
Iowa	9,757,950	7,241,804	3,517,926	493.8	3,399,846	1,595,390	107.3	2,921,160	1,273,329	86.2	3,371,305
Kansas	10,005,211	3,231,854	1,610,483	664.5	5,287,970	2,617,534	367.6	4,567,954	2,283,963	182.5	3,493,231
Kentucky	6,961,271	2,255,639	1,102,787	150.7	1,362,578	681,294	28.7	2,003,563	1,001,781	61.4	4,175,409
Louisiana	5,387,420	1,855,513	925,033	66.1	810,147	405,072	28.1	1,210,494	545,330	30.7	3,511,985
Maine	3,299,867	1,919,415	931,686	58.9	888,898	444,449	22.4	921,720	460,860	31.0	1,462,873
Maryland	3,094,808				1,256,417	628,173	19.6	563,067	281,533	8.4	2,185,102
Massachusetts	5,255,300	333,935	166,968	3.1	4,322,180	2,161,090	20.3	487,090	243,545	2.3	2,683,697
Michigan	11,562,296	9,230,185	4,561,003	332.3	4,938,669	2,468,431	123.7	3,357,350	1,678,050	86.7	2,854,812
Minnesota	10,344,485	8,193,397	3,847,653	532.8	2,515,487	1,255,731	133.2	1,777,230	888,615	70.9	4,352,486
Mississippi	6,635,344	4,640	2,320		1,211,700	605,850	79.4	3,565,100	1,781,440	153.4	4,245,734
Missouri	11,479,090	4,104,251	2,043,199	448.9	7,883,065	3,928,253	265.3	2,662,468	1,320,427	113.1	4,187,210
Montana	7,744,061	4,134,820	2,314,535	409.3	2,226,576	1,246,660	147.1	1,222,888	646,873	63.4	3,535,993
Nebraska	7,809,353	3,013,595	1,504,069	182.1	2,616,178	1,313,671	276.7	1,654,074	827,018	217.1	4,164,595
Nevada	4,821,864	1,664,266	1,435,963	272.2	1,674,670	1,446,669	54.4	9,600	8,300	.5	1,930,932
New Hampshire	1,343,750	841,062	406,141	24.8	192,933	96,014	2.3				1,341,595
New Jersey	5,054,295	2,466,551	1,233,124	34.6	2,257,052	1,051,231	24.9	48,350	24,175		2,745,764
New Mexico	6,030,708	3,656,928	2,244,783	300.0	1,686,671	1,025,817	95.3	1,080,108	658,336	71.4	2,101,772
New York	18,565,567	8,074,339	3,976,459	167.7	15,937,284	7,637,642	252.9	1,924,500	647,250	27.4	6,304,216
North Carolina	8,877,837	2,492,655	1,242,861	313.0	3,870,530	1,931,965	262.1	2,558,740	1,099,820	117.7	4,603,191
North Dakota	5,914,683	193,080	103,375	.3	191,740	101,206	.1	487,132	487,040	79.8	5,223,062
Ohio	13,771,548	2,481,216	1,236,370	44.3	6,123,269	2,939,631	64.5	2,748,680	1,349,840	28.6	8,245,707
Oklahoma	8,880,547	2,714,308	1,410,154	93.4	2,427,856	1,273,571	86.0	2,136,901	1,122,921	80.4	5,073,901
Oregon	6,182,079	3,002,674	1,807,515	112.8	3,075,531	1,836,922	129.4	1,224,945	681,092	30.0	1,856,550
Pennsylvania	16,129,304	6,677,091	3,316,059	111.3	8,391,644	4,187,994	120.6	4,015,953	1,937,511	54.7	6,688,241
Rhode Island	1,843,750	213,888	106,944	3.4	678,336	339,168	5.6	190,077	92,864	2.1	1,304,774
South Carolina	5,103,525	576,230	219,400	50.9	4,440,530	1,781,020	278.1	716,752	272,302	68.0	2,830,803
South Dakota	6,162,747	1,385,603	778,010	188.6	127,860	80,869	36.5	1,142,696	628,344	91.7	4,675,525
Tennessee	7,949,380	2,260,730	1,128,235	98.8	1,055,616	527,803	36.2	1,400,508	700,259	53.0	5,593,083
Texas	23,506,431	12,553,750	6,253,927	696.2	9,111,160	4,513,168	616.3	5,019,684	2,478,595	294.6	10,260,742
Utah	4,274,740	2,090,475	1,462,744	138.1	1,126,283	811,541	82.5	131,050	90,226	12.8	1,910,229
Vermont	1,843,750	1,309,726	647,508	62.9	801,645	373,910	23.1	250,299	125,070	9.2	697,262
Virginia	6,887,569	2,834,126	1,414,222	112.8	2,844,932	1,422,463	125.4	1,628,433	814,217	48.8	3,236,667
Washington	5,907,615	3,946,453	2,070,727	155.1	2,040,606	1,070,871	79.5	458,276	230,060	3.7	2,535,957
West Virginia	4,107,201	863,018	431,486	42.3	890,456	445,216	23.9	594,525	297,263	10.9	2,933,237
Wisconsin	9,197,557	3,862,454	1,833,583	160.6	4,250,178	2,070,628	134.0	476,190	237,230	14.1	5,056,115
Wyoming	4,722,322	2,857,375	1,750,138	360.8	1,803,756	1,105,400	180.0	1,246,750	770,480	117.5	1,096,304
Puerto Rico	625,000										625,000
Hawaii	1,843,750	29,953	14,542	.8	783,072	388,545	14.5	111,710	55,520	2.9	1,385,143
TOTALS	368,750,000	146,855,669	76,406,844	8,071.6	147,143,838	76,167,844	5,413.5	70,934,772	36,314,870	2,828.3	179,860,442

1/ Apportionments for Fiscal Years 1936 to 1938, Inclusive.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MARCH 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,312,723	$ 3,297,736	109.2	$ 771,036	$ 771,036	29.1				$ 82,342
Arizona	2,569,841	2,554,786	2,197,348	163.0	627,373	330,201	32.9				42,291
Arkansas	3,352,061	2,630,094	2,611,333	278.2	701,359	700,085	80.9				40,643
California	7,747,928	5,778,244	5,603,006	228.4	2,064,739	1,969,697	26.8	$ 267,700	$ 61,380	9.1	113,845
Colorado	3,395,263	1,816,571	1,760,386	98.2	153,510	153,494	7.0				1,481,383
Connecticut	1,418,709	419,092	400,115	4.3	552,984	527,217	10.7	206,152	190,569	5.4	300,808
Delaware	900,310	497,551	476,418	47.0	355,632	276,827	19.6				147,065
Florida	2,597,144	1,592,710	1,588,873	75.1	943,377	943,377	23.9				64,893
Georgia	4,988,967	501,931	491,984	29.4	975,075	975,075	64.9	445,112	445,112	36.3	3,076,796
Idaho	2,222,747	2,222,611	2,132,607	182.9	56,927	56,927	2.7				33,213
Illinois	8,694,009	7,456,674	7,397,521	423.8	1,262,470	1,262,392	47.3				34,096
Indiana	4,941,255	2,865,760	2,714,272	122.5	2,187,703	2,187,703	106.0	37,924	37,924	6.3	1,355
Iowa	4,991,664	3,199,450	3,083,008	368.4	2,028,446	1,850,040	157.0	65,891	49,614	2.9	9,002
Kansas	4,994,975	2,636,707	2,617,489	270.8	2,390,421	2,362,181	104.3	15,305	15,305	.8	
Kentucky	3,726,271	2,677,849	2,645,394	311.0	662,136	662,136	34.0	238,799	238,790	6.7	179,951
Louisiana	2,890,429	1,324,897	1,169,740	95.0	1,693,568	1,549,394	71.2	171,682	157,053	10.0	14,242
Maine	1,676,799	1,286,971	1,279,028	57.6	317,194	317,194	14.3	62,100	62,100	2.1	18,483
Maryland	1,750,738	378,205	376,029	17.6	564,058	564,058	10.9	468,914	369,484	12.6	441,168
Massachusetts	3,262,885	216,783	216,783	2.5	2,684,380	2,331,020	15.9	782,404	409,682	.4	305,400
Michigan	6,301,414	6,233,066	6,006,108	287.9	291,871	246,871	4.8	5,766	4,650		43,785
Minnesota	5,277,145	5,153,857	4,382,805	815.6	1,147,311	743,326	85.5	137,137	120,248	2.3	30,766
Mississippi	3,457,552	2,074,178	2,070,270	154.2	1,351,963	1,350,593	80.5				36,689
Missouri	6,012,652	4,072,175	4,035,972	730.4	2,043,541	1,858,824	46.3	38,755	36,659	1.1	81,197
Montana	3,676,415	3,355,609	3,352,332	185.1	245,028	245,028	10.2	9,222	9,222		69,834
Nebraska	3,870,739	2,422,010	2,372,133	264.7	1,138,567	1,137,063	98.8	120,677	120,677	6.5	240,866
Nevada	2,243,074	1,935,300	1,879,205	89.1	342,430	327,430	13.7	2,738	2,738	.3	33,701
New Hampshire	945,225	614,211	588,225	26.7	218,593	216,295	8.1	71,396	70,636	2.8	70,070
New Jersey	3,129,805	667,446	664,446	14.0	2,310,652	2,297,497	16.9	137,412	128,443	6.1	39,420
New Mexico	2,871,397	2,281,478	2,279,516	179.2	375,715	375,715	13.9	105,829	105,829	8.2	110,337
New York	11,046,377	9,110,008	8,683,859	133.8	2,019,260	2,019,260	34.1	305,778	305,778	2.4	37,480
North Carolina	4,720,173	2,730,571	2,697,876	201.2	1,860,128	1,860,128	81.3	124,940	89,000	8.8	73,169
North Dakota	2,867,245	1,529,156	1,526,315	222.2	952,048	948,613	83.3	144,327	144,271	28.2	248,046
Ohio	7,670,815	3,049,995	2,985,685	138.0	4,162,237	4,114,802	134.0	451,150	448,630	30.5	121,698
Oklahoma	4,580,670	2,878,675	2,842,131	282.7	1,391,183	1,378,524	100.2	205,712	205,652	13.6	154,363
Oregon	3,038,642	2,054,423	2,035,590	149.3	1,199,861	838,427	8.9	116,374	116,374	9.5	48,251
Pennsylvania	9,347,797	1,817,187	1,735,150	90.9	2,401,676	2,343,951	64.2	2,629,144	2,506,895	86.6	2,761,802
Rhode Island	989,208	1,054,850	973,538	18.8	9,520	9,520					6,150
South Carolina	2,702,012	1,236,313	1,167,394	119.0	1,104,867	1,080,086	103.3	242,781	242,561	13.4	211,971
South Dakota	2,976,454	1,952,709	1,951,085	383.0	744,347	744,347	80.5	184,020	184,020	29.8	97,002
Tennessee	4,192,460	2,200,158	2,193,276	92.0	1,260,081	1,260,081	42.5	382,822	382,822	2.9	356,281
Texas	11,989,350	11,693,999	10,730,751	1,066.0	1,295,888	1,136,373	41.0	76,061	60,475	6.0	61,751
Utah	2,067,154	1,621,092	1,472,140	163.5	431,344	399,980	26.2	76,141	76,141	3.4	118,893
Vermont	924,306	831,474	734,053	20.6	219,087	160,687	1.6	19,962	19,830	.9	9,736
Virginia	3,652,667	2,874,432	2,807,876	917.8	577,638	576,978	100.7	80,891	69,370	19.1	198,443
Washington	3,026,161	2,621,613	2,356,635	157.7	775,869	669,526	6.7				
West Virginia	2,231,412	645,820	642,323	30.7	1,652,810	1,529,211	63.8	77,060	57,380	2.9	2,497
Wisconsin	4,823,884	4,913,978	4,344,446	328.1	509,195	368,403	15.1				111,035
Wyoming	2,219,155	1,604,518	1,587,722	121.7	600,269	581,401	29.4				50,032
District of Columbia	949,496	949,496	949,496	8.8							
Hawaii	926,033	549,266	530,091	7.3	369,741	366,432	10.0				29,511
TOTALS	195,000,000	130,098,672	124,637,508	10,284.9	53,995,108	50,975,426	2,264.9	8,508,078	7,545,314	377.9	11,841,752

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MARCH 31, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 2,379,256	$ 2,379,256	33	1	6	$ 1,457,214	$ 1,457,214	15			$ 6,220	$ 6,220			6	$ 191,927
Arizona	1,256,099	825,527	821,292	9			348,834	337,389	4								97,418
Arkansas	3,574,060	1,460,695	1,455,772	32	4		1,885,363	1,882,858	23	2	2	189,883	189,613	1		28	45,817
California	7,486,362	5,138,522	4,963,742	29	8		2,291,657	2,223,580	15			95,400	95,400	7			203,639
Colorado	2,631,567	1,234,005	1,206,527	21	1		939,941	939,940	8								485,100
Connecticut	1,712,684	73,479	73,479		1		779,991	758,630	5		1	76,314	76,314		1		804,261
Delaware	418,239	130,000	130,000	1													288,239
Florida	2,827,883	1,587,949	1,585,639	16	5		799,954	799,611	12	1		178,000	178,000	2			264,634
Georgia	4,895,949	30,729	29,237		2		852,574	852,574	18	3		317,428	317,428	7		38	3,696,710
Idaho	1,674,479	904,725	898,260	14	1	3	377,057	376,944	5	1	4	24,368	24,368			14	374,907
Illinois	10,307,184	4,012,232	4,011,270	44	1		5,470,293	5,367,093	28	6		850,600	850,600	4			78,221
Indiana	5,111,096	1,709,724	1,579,346	20	9		3,556,105	3,519,505	22	3	162						12,246
Iowa	5,600,679	2,640,027	2,583,088	65	7	1	2,826,646	2,746,787	40	2		257,634	236,880	2		7	33,924
Kansas	5,246,258	1,459,190	1,458,941	22		3	3,838,540	3,763,697	36	1		6,549	6,549			2	17,071
Kentucky	3,672,387	878,563	878,563	12	2		1,916,291	1,626,559	10	3		534,480	534,480	4	3		632,784
Louisiana	3,213,467						1,718,021	1,718,021	18	2		801,791	757,074	5	2		738,372
Maine	1,426,861	485,563	484,640	12		2	584,695	584,103	8			158,534	158,534	1	1		199,584
Marvland	2,061,751	358,388	358,388	3	3		518,504	518,504	3		19	600,004	577,536	2	2		607,323
Massachusetts	4,210,833	888,240	888,240	7	2		2,144,993	2,144,993	16	2		249,991	249,991	1			927,609
Michigan	6,765,197	3,331,462	3,274,451	34	4		3,454,277	3,454,277	11	4							36,469
Minnesota	5,395,441	2,835,355	2,765,325	59	9	37	2,509,840	2,482,579	26	3	3	58,428	58,428		1	1	89,109
Mississippi	3,241,475	1,040,286	1,040,138	28	3	1	1,550,977	1,550,977	24	1		127,270	127,270	4	1	9	523,090
Missouri	6,142,153	651,049	651,049	12			5,337,317	5,141,073	36	1		367,252	253,300	1		2	96,731
Montana	2,722,327	2,495,057	2,494,551	35	6		178,582	178,582	2								49,194
Nebraska	3,556,411	1,759,812	1,757,577	61	1	2	1,414,900	1,414,900	16	2	4	336,118	336,118	4		18	47,846
Nevada	887,260	690,198	690,198	7	3		211,658	193,432	1			3,630	3,630			5	
New Hampshire	822,484	341,748	341,748	3	3		235,462	235,417	3	1	1	168,326	168,326	2			76,993
New Jersey	3,983,826	760,036	760,036	6	2		2,086,690	2,075,645	13	2		490,706	490,706	2	3		657,440
New Mexico	1,725,286	1,053,490	1,053,475	13	1		673,090	670,400	5								1,410
New York	13,577,189	2,393,473	2,388,673	12	9		9,754,281	9,498,235	28	34		655,900	655,900	1	3		1,034,381
North Carolina	4,823,958	1,417,907	1,417,907	19	9		2,270,126	2,250,575	27	8	111	456,100	456,100	4			699,376
North Dakota	3,207,473	489,481	489,402	16			2,003,740	2,000,183	27	4		335,570	335,570	10			382,318
Ohio	8,439,897	342,256	276,763	2	1		5,435,148	5,277,147	35	5	2	2,054,920	1,897,862	17	1	1	988,124
Oklahoma	5,004,711	1,631,466	1,631,396	31	3		1,868,321	1,868,321	22	4		166,996	166,996	3	1	3	1,337,998
Oregon	2,334,204	878,991	878,452	8	5	2	1,491,605	1,410,005	8	1							45,748
Pennsylvania	11,483,613	1,964,345	1,760,257	26	8		7,501,793	6,993,014	45	8	9	879,478	861,678	4	4		1,868,664
Rhode Island	699,691	624,125	623,059	4	1		61,626	61,626		2							15,006
South Carolina	3,059,956	587,872	579,006	14	2	3	1,402,699	1,392,505	26	8		432,825	432,825	3	1	45	655,620
South Dakota	3,249,086	739,942	739,942	22	1	14	1,496,475	1,496,475	31	4	11	438,165	438,165	10		17	574,504
Tennessee	3,903,979	551,263	551,253	11	1	17	2,248,310	2,248,310	25	2	2	526,470	526,470	6		7	577,947
Texas	10,855,982	4,679,868	4,676,065	79	12	1	5,261,403	5,257,162	50	2	5	510,622	510,616	1		171	412,140
Utah	1,230,763	89,918	88,528	1	1	1	1,107,705	1,107,705	16								34,529
Vermont	729,857	499,386	493,786	7	5	14	227,863	187,700	3	1	2	52,138	48,371		1	4	
Virginia	3,774,287	1,617,449	1,546,584	30	7	6	949,070	910,071	11	9	2	284,562	284,562	9	1	12	1,033,070
Washington	3,095,041	1,048,408	1,043,576	15	6	7	1,791,849	1,791,424	8	5		295,031	260,041		1	2	
West Virginia	2,677,937						1,692,633	1,692,631	20	3	1	269,672	269,672	6		4	715,633
Wisconsin	5,022,683	2,020,579	1,988,506	25	4		2,935,206	2,843,954	12	3							190,223
Wyoming	1,360,841	540,621	540,513	7			651,776	651,775	5								168,553
Dist. of Columbia	410,804	170,643	170,643	2			254,921	226,161	1								14,000
Hawaii	453,703	170,404	170,389	2			351,976	283,314	3								
TOTALS	196,000,000	63,613,714	62,668,928	931	154	120	100,717,992	98,463,577	826	143	341	13,257,375	12,841,593	123	27	396	22,025,902

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

DEPARTMENT BULLETINS

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

TECHNICAL BULLETINS

No. 265T . . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

No. 191 . . . . Roadside Improvement. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF MARCH 31, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8.370.133	$ 4.259.842	$ 15,671,454	$ 8,301,315	$ 3,813,459	770.8	$ 171.795	$ 55,923	$ 115,872	5.8		$ 285,822	1.5	$ 12,895	$ 44,689
Arizona	5.211,960	2.641,935	8,984,153	5,203,675	2,605,035	543.2	24,073		24,073		$ 5,574	810	.1	2,711	12,017
Arkansas	6.748,335	3,428,049	10,968,973	6,627,424	3,371,441	619.9	130.731	80,459	49,922	6.5	33,944	4,200	10.9	6,508	2,486
California	15,607,354	7,932,206	30,647,428	15,607,354	7,741,183	758.7	115,605		115,553						75,471
Colorado	6,874,530	3,486,006	11,256,388	6,874,530	3,459,042	633.2	6,362		6,360						20,604
Connecticut	2,865,740	1,454,868	4,516,913	2,758,269	1,315,170	74.0	59,618	59,618		.1				47,854	139,697
Delaware	1,819,088	923,395	2,782,833	1,818,804	916,230	130.0								284	7,165
Florida	5,231,834	2,661,343	9,010,010	5,175,534	2,419,763	307.3	289,764	56,300	233,464	2.9					8,116
Georgia	10,091,185	5,113,491	13,270,465	9,193,853	3,203,526	754.4	1,143,612	408,690	734,922	73.4	131,952	166,754	10.8	356,691	1,008,288
Idaho	4,486,249	2,277,486	7,077,041	4,477,339	2,153,743	501.4	85,967		83,807	.1		1,886		8,909	38,050
Illinois	17,570,770	8,921,401	26,441,293	17,393,251	7,957,127	716.7	1,141,513	129,245	885,007	13.3	17,560	7,100	7.5	30,714	72,167
Indiana	10,037,843	5,088,963	15,561,813	9,892,557	4,818,774	482.7	227,844	129,083	98,761	2.4		61,728		16,203	109,700
Iowa	10,055,660	5,118,361	15,581,748	10,055,660	4,920,929	1,222.3	209,734		196,600	5.0					832
Kansas	10,089,604	5,117,675	15,426,562	10,089,600	5,009,852	1,132.3	135,923		99,512	13.2				4	8,311
Kentucky	7,517,359	3,818,311	12,185,269	7,504,603	3,719,464	812.7	69,829		69,829	2.8	10,217	15,467		2,539	13,551
Louisiana	5,828,591	2,963,932	9,145,365	5,763,651	2,712,794	259.5	215,480		215,459	11.5				64,940	35,679
Maine	3,369,917	1,711,586	5,248,303	3,346,497	1,671,457	193.5	43,100	15,000	26,540	1.4				8,420	13,589
Maryland	3,564,527	1,810,058	5,659,409	3,475,617	1,103,886	152.2	68,343		68,343	1.5		285,453	7.3	88,910	352,377
Massachusetts	6,597,100	3,350,474	10,399,518	6,552,734	3,094,281	115.5						91,944	.1	44,367	164,249
Michigan	12,736,227	6,452,568	20,697,260	12,736,227	6,304,754	768.1	96,788		96,788			40,000	.4		11,026
Minnesota	10,656,569	5,425,551	16,208,196	10,510,789	4,850,678	1,641.9	372,262	58,110	268,494	7.2		34,800	.7	87,669	271,579
Mississippi	6,978,675	3,540,227	12,756,603	6,788,308	2,982,044	723.8	655,825	142,960	434,238	21.7	9,360	12,131	.4	38,047	51,814
Missouri	12,180,306	6,173,740	18,180,504	12,088,271	4,995,054	1,441.2	1,125,698		1,121,765	6.3				92,035	56,920
Montana	7,439,748	3,769,734	11,764,351	7,425,476	3,655,874	1,058.4	56,546		56,546					14,272	57,314
Nebraska	7,828,961	3,964,364	12,916,662	7,812,918	3,692,772	1,024.7	251,072		251,072	27.1				16,043	20,520
Nevada	4,545,917	2,302,356	7,068,575	4,545,917	2,271,616	758.8	21,979		21,979			1,650			7,111
New Hampshire	1,903,839	969,462	2,999,744	1,904,951	949,520	78.3	4,169		4,169					4,888	15,773
New Jersey	6,346,039	3,220,879	9,096,652	6,045,652	2,452,166	85.9	1,301,908	204,338	663,430	10.9	15,734	9,212	.2	80,316	96,071
New Mexico	5,792,935	2,941,700	8,898,727	5,749,151	2,910,898	749.9	9,681		9,681			7,047	.2	43,784	14,074
New York	22,330,101	11,327,921	39,513,163	21,861,146	10,475,519	821.9	1,344,462	457,434	687,949	3.4	5,522	112,200		5,999	52,253
North Carolina	9,522,293	4,840,941	14,958,870	9,210,235	4,516,899	1,345.8	553,051	291,910	261,141	21.5				20,147	62,901
North Dakota	5,804,448	2,938,967	8,583,233	5,582,621	2,219,879	2,121.9	253,154	132,456	120,698	21.2	50,040	178,190	29.2	39,332	420,199
Ohio	15,434,592	7,865,012	24,762,040	15,378,758	7,526,175	792.1	281,771	40,500	212,354	4.0	58,056	78,000	4.2	7,278	48,483
Oklahoma	9,216,798	4,685,180	14,741,691	9,214,658	4,430,547	806.6	196,489		196,489	.4		31,465		2,141	26,679
Oregon	6,106,896	3,097,814	9,956,493	6,048,081	2,944,641	468.0	64,223		64,223	.8	55,000	16,694	.7	3,815	72,257
Pennsylvania	18,891,004	9,590,788	28,996,008	18,533,017	8,905,563	1,054.0	534,536	192,542	321,211	5.8	149,635	319,659	10.0	15,809	44,355
Rhode Island	1,998,708	1,014,572	3,150,270	1,998,708	1,012,094	89.1	2,478		2,478						
South Carolina	5,459,165	2,770,954	7,970,188	5,206,684	2,435,279	618.3	370,183	161,850	208,003	9.7	40,000	92,710	9.1	50,631	34,962
South Dakota	6,011,479	3,047,643	9,270,234	5,834,795	2,859,580	1,571.6	229,558	77,600	151,958	29.5	55,660	7,630	7.2	44,024	28,475
Tennessee	8,492,619	4,302,991	13,667,901	8,490,706	4,135,256	492.7	140,180		113,164	3.6		46,727	5.9	1,913	7,845
Texas	24,244,024	12,291,253	37,970,837	24,186,074	11,873,448	2,780.0	206,314	14,797	191,197	2.0		193,334	8.0	43,150	33,274
Utah	4,194,708	2,132,691	7,430,826	4,192,008	2,132,691	590.9								2,700	
Vermont	1,867,573	948,007	3,166,359	1,867,452	940,847	141.0								121	7,160
Virginia	7,416,757	3,765,387	11,572,376	7,290,407	3,357,346	617.0	319,077	83,691	229,754	31.5	6,735	29,892	1.5	35,924	148,395
Washington	6,115,867	3,106,412	9,397,793	6,112,042	3,026,509	302.7	46,596		46,596			14,823	.3	3,825	18,484
West Virginia	4,474,234	2,230,335	6,464,966	4,342,391	1,782,567	212.3	375,487	54,892	297,549	9.4		66,425	.7	76,951	133,794
Wisconsin	9,724,881	4,941,837	15,443,236	9,724,881	4,883,612	619.7	14,415		14,415			635			38,175
Wyoming	4,501,327	2,287,712	6,903,736	4,451,922	2,235,553	1,037.7	37,731	11,728		2.6	4,770		.3	32,906	52,159
District of Columbia	1,918,469	973,842	2,887,584	1,918,469	968,979	22.3						4,863			
Hawaii	1,871,062	949,778	2,779,059	1,857,812	362,218	51.8	552,762		549,594	4.1		14,000		13,250	23,966
TOTALS	394,000,000	200,000,000	630,009,125	389,022,194	184,102,734	35,069.2	13,557,688	2,859,126	9,680,959	362.6	649,759	2,233,251	117.2	1,468,921	3,983,056

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE EFFECT OF CURING CONDITIONS ON STRENGTH OF CONCRETE TEST SPECIMENS CONTAINING BURNT CLAY AGGREGATES

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by W. F. KELLERMANN, Associate Materials Engineer

THE TREND toward increased span lengths on highway bridges has stimulated efforts to decrease the dead load caused by the weight of the bridge floor. Results of a study of the relative strengths of test specimens of concrete bridge floor slabs made of several materials and placed by each of several methods have been published.[1]

Included among the concrete aggregates used was the artificial, lightweight aggregate known as "Haydite." The tests showed that there was a progressive retrogression in flexural strength for the Haydite concrete specimens for ages exceeding 90 days, whereas concrete made with siliceous gravel, crushed limestone, and quartz sand aggregates, cured under presumably the same conditions, continued to gain in strength. In view of the probability that this behavior of the Haydite specimens might be related to the moisture-retaining qualities of the aggregate, it was decided to conduct further studies of Haydite concrete specimens. A supplementary series of laboratory tests under controlled curing conditions is discussed in this report.

Two series of concrete specimens, one containing fine and coarse Haydite as total aggregate and the other a natural sand and crushed limestone, were made and tested at various ages up to and including 1 year. In addition, a limited number of specimens made with Haydite aggregates were tested at an age of 428 days, and corresponding specimens made with natural aggregates were tested at 409 days. The mix used with the natural aggregates was 1 : 2.1 : 3.9 by dry loose volume (1 : 2 : 3.5 by dry-rodded volume). The mix used with the Haydite aggregates was 1 : 1.5 : 3 by dry loose volume, with 3 pounds of "Celite" added per sack of cement.

The latter mix was the same as that used in the bridge slab tests where the retrogression in flexural strength was noted. However, the Haydite used in the previous tests came from Jackson County, Missouri, while that used in these tests came from Erie County, New York. Though the materials were from different sources, both were manufactured by the same process and were submitted by the manufacturer as being representative of the type. The natural sand came from the Potomac River at Washington, the limestone from Martinsburg, W. Va. Physical characteristics of the aggregates are given in table 1, and the mix data are shown in table 2.

MIXING AND TESTING PROCEDURES DESCRIBED

Because dried Haydite aggregate absorbs water rapidly, it was decided to presaturate all aggregates and use them in this condition. The procedure followed was to dry the materials in an oven, weigh out the fine and coarse aggregates required for each batch in a watertight container, and immerse them for 48 hours in a measured quantity of water before mixing. By this procedure the total water used was accurately determined, yet thorough presaturation of the aggregates was accomplished. However, the net water-cement ratio reported for the Haydite concrete is only approximate, because of the difficulty of determining accurately the amount of water absorbed by the aggregates.

TABLE 1.—*Grading and physical properties of aggregates*

	Haydite fine aggregate	Potomac River sand	Haydite coarse aggregate	Martinsburg limestone
Retained on:	*Percent*	*Percent*	*Percent*	*Percent*
1½-inch sieve				
1-inch sieve				20
¾-inch sieve				49
½-inch sieve			17	74
⅜-inch sieve			60	87
4-mesh sieve		5	98	100
8-mesh sieve	14	24	100	100
16-mesh sieve	48	41	100	100
30-mesh sieve	67	59	100	100
50-mesh sieve	78	86	100	100
100-mesh sieve	84	97	100	100
Fineness modulus	2.91	3.12	6.58	7.36
Physical properties:				
Specific gravity [1]	1.63	2.53	1.42	2.71
Absorption [2]—percent	11.4	2.1	7.8	0.13
Weight (dry loose) per cubic foot—pounds	57	101	45	90
Voids—percent	44	36	49	47

[1] Bulk specific gravity, A. S. T. M. definition E 12—27.
[2] 70° cone method of Bureau of Public Roads.

TABLE 2.—*Data on mixes used in making concrete specimens*

Aggregates		Proportion by dry loose volume	Celite per sack of cement	Water content per sack of cement		Consistency flow [1]	Weight	Cement factor
Coarse	Fine			Total	Net			
			Lb.	*Gal.*	*Gal.*		*Lb. per cu. ft.*	*Sacks per cu. yd.*
Haydite	Haydite	1:1.5:3.0	3	8.4	5.9	140	104	7.2
Limestone	Potomac River sand	[2] 1:2.1:3.9		6.4	5.8	165	151	5.7

[1] Fifteen ½-inch drops in 10 seconds.
[2] Proportions by dry-rodded volume 1:2:3.5.

The concrete was mixed with shovels in watertight pans, each batch being of sufficient size to make one flexure specimen (6- by 6- by 21-inch beam) or two 6- by 12- inch cylinders. After 1 day in the molds and 6 days in the moist room, the flexural specimens were subjected to the curing conditions indicated in table 3, and the compression specimens to the curing conditions shown in table 4. Moist curing was accomplished by storing the

[1] The Effect of Materials and Methods of Placing on the Strength and Other Properties of Concrete Bridge Floor Slabs, by L. W. Teller and G. W. Davis. PUBLIC ROADS, vol. 12, no. 10, December 1931.

specimens in the moist room under standard conditions of humidity and temperature. The final wet curing consisted of immersion in water at laboratory temperatures prior to test.

TABLE 3.—*Curing conditions for concrete beam specimens*

Group no.	Period of curing				Age at test
	Initial moist curing	Air	Oven at 170° F.	Water at 70° F. prior to test	
	Days	*Days*	*Days*	*Days*	*Days*
1	7				7
2	7	21			28
3	28				28
4	7	173			180
5	7	166		7	180
6	7	169		4	180
7	7	170		3	180
8	7	171		2	180
9	7	172		1	180
10	173	7			180
11	172	1	7		[1] 180
12	180				180
13	7	353			360
14	7	346		7	360
15	28	332			360
16	180	180			360
17	270	90			360
18	28	325		7	360
19	180	173		7	360
20	270	83		7	360
21	353	7			360
22	352	1	7		[1] 360
23	359	1			360
24	358	1	1		[1] 360
25	360				360
26	[2] 360	1	[3] 67		[1] 428
	[4] 360	1	[3] 48		[1] 409
27	[2] 428				428
	[4] 409				409
28	[2] 7	421			428
	[4] 7	402			409

[1] Oven-dried specimens cooled in air of laboratory for 1 day prior to test.
[2] Data for beams having Haydite aggregates.
[3] Dried in oven to constant weight prior to test.
[4] Data for beams having natural sand and crushed limestone aggregates.

TABLE 4.—*Curing conditions and crushing strengths of cylinders*

Group no.	Age at test	Curing		Coarse aggregate				Strength ratio, Haydite concrete to limestone concrete
		Moist room	Air	Haydite		Limestone		
				Crushing strength[1]	Coefficient of variation	Crushing strength[1]	Coefficient of variation	
	Days	*Days*	*Days*	*Lb. per sq. in.*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*
1	7	7		2,300	3.5	2,280	3.8	101
2	28	7	21	3,460	6.2	3,510	2.1	99
3	28	28		3,970	4.6	4,040	5.4	98
4	180	7	173	5,180	1.4	4,470	3.7	116
12	180	180		6,490	5.6	5,270	1.8	123
13	360	7	353	5,230	4.4	4,250	4.4	123
25	360	360		6,850	1.9	5,410	3.2	127
Average				4,780	3.9	4,180	3.5	114

[1] Each value represents the average of tests on 5 specimens.

Air curing was accomplished by storing the specimens in air in a frame structure which was heated only enough to keep the specimens from freezing. Twenty-four hours before test the air-cured specimens were moved to the laboratory in order to attain laboratory temperatures, and the final day was counted as part of the curing period. Oven-dried specimens were heated in an electric oven maintained at a temperature of 170° F. ± 5° F. and were brought to the laboratory 24 hours before test in order to cool. Each value given in tables represents the average of tests on five specimens in practically all cases.

The compression specimens were standard 6- by 12-inch cylinders; the flexural specimens were 6- by 6- by 21-inch beams. Flexure specimens were weighed at the time they were fabricated, when they were removed from the molds, at each time the curing conditions were changed, and at the time of test. Thus it was possible to calculate the percentage of water remaining in each specimen at each change of curing condition and at test.

The flexure specimens were tested as simple beams on an 18-inch span with a side as molded, in tension. They were loaded at the third points with the apparatus described in PUBLIC ROADS (vol. 13, no. 11), January 1933. The compression specimens were tested in accordance with standard methods.

The percentage of original mixing water remaining in the flexural specimens at the periods indicated is shown in table 5. Similar data are not available for the cylinders, because it was necessary to cap them, thereby changing the weight as well as the composition of the individual specimens. The average flexural strength for each group tested is given in table 6; table 4 gives similar data as regards crushing strength.

HAYDITE CONCRETE RELATIVELY SLOW IN ABSORBING AND LOSING MOISTURE

Table 5 shows the amounts of water remaining in the beam specimens at various periods, expressed as percentages of the original mixing water. The results show that the Haydite concrete lost approximately 7 percent of its mixing water during the first 24 hours. After 6 days curing in the moist room the specimens had reabsorbed moisture until the water content averaged about 95 percent of the original amount. After 28 days the percentage had increased to about 97 percent, and complete saturation was reached at 180 days. For the limestone concrete the percentage of original mixing water at 24 hours averaged about 91 percent. After curing in the moist room for 6 days the water content was about 95 percent, and complete saturation was attained at 28 days. Complete saturation is considered to have been reached when the water content of the specimen was 100 percent of the original amount of water used.

The amounts of moisture lost by specimens cured in the moist room for 6 days, followed by curing in air for varying periods, are shown for groups 2, 4 to 9 inclusive, 13, 14, and 28 in table 5. It will be observed that for both aggregates the moisture loss reached a maximum after about 6 months' curing in air. Increasing the curing period beyond 6 months (groups 13, 14, and 28) appeared to have negligible effect insofar as drying was concerned.

Data for groups 5 to 9, inclusive, show the effect of immersing the specimens from 1 to 7 days in water after approximately 6 months curing in air. The initial curing period was 6 days in the moist room in each case. It will be observed that after 7 days of immersion (group 5) the limestone concrete had reabsorbed a considerably larger proportion of its original mixing water than had the Haydite concrete. Moreover, the rate of gain in weight after the first 24 hours in water appeared to be considerably less for the Haydite concrete.

Comparing groups 5 and 14 we find that for the limestone concrete, 7 days' curing in water was sufficient time for the specimens to regain all the moisture lost during the 6-month or 1-year curing in air. This was not true of the Haydite concrete, the amount of water reabsorbed after 7 days curing in water being substantially less than the amount present at the time the specimens were subjected to air curing.

TABLE 5.—*Percentages of original mixing water remaining in concrete specimens after curing*

CONCRETE BEAM SPECIMENS CONTAINING HAYDITE AGGREGATES

Group no.	Percentage of original mixing water remaining after curing in—												
	Molds for 24 hours	Moist room		Air		Oven		Water		Laboratory air		At test
		Period	Water content	Period	Water content	Period	Water content	Period	Water content	Period	Water content	
	Percent	*Days*	*Percent*	*Days*	*Percent*	*Days*	*Percent*	*Days*	*Percent*	*Days*	*Percent*	*Percent*
1	93	6	94									94
2	93	6	96	20	73					1	71	71
3	92	27	96									96
4	94	6	96	172	62					1	62	62
5	93	6	96	166	62			7	82			82
6	92	6	94	169	60			4	81			81
7	93	6	95	170	62			3	81			81
8	92	6	94	171	59			2	80			80
9	94	6	97	172	62			1	81			81
10	93	172	100	6	95					1	95	95
11	94	171	100			7	63			1	64	64
12	93	179	100									100
13	93	6	96	352	61					1	61	61
14	93	6	94	346	60			7	84			84
15	94	27	98	331	70					1	70	70
16	93	179	100	179	83					1	84	84
17	93	269	100	89	88					1	88	88
18	93	27	96	325	69			7	87			87
19	93	179	98	173	83			7	92			92
20	93	269	99	83	88			7	95			95
21	94	352	101	6	97					1	96	96
22	96	351	101			7	74			1	75	75
23	93	358	100							1	97	97
24	94	357	101			1	90			1	90	90
25	94	359	100									100
26	94	359	101			67	29			1	30	30
27	95	427	100									100
28	93	6	96	352	60					69	58	58

CONCRETE BEAM SPECIMENS CONTAINING NATURAL SAND AND CRUSHED LIMESTONE

1	92	6	96									96
2	90	6	93	20	76					1	75	75
3	93	27	100									100
4	92	6	96	172	66					1	66	66
5	91	6	96	166	67			7	95			95
6	92	6	95	169	65			4	95			95
7	90	6	94	170	66			3	92			92
8	91	6	96	171	65			2	91			91
9	89	6	93	172	64			1	88			88
10	94	172	103	6	97					1	96	96
11	92	171	101			7	52			1	53	53
12	90	179	101									101
13	92	6	96	352	67					1	67	67
14	92	6	96	346	67			7	97			97
15	92	27	100	331	70					1	70	70
16	92	179	100	179	80					1	79	79
17	91	269	101	89	86					1	85	85
18	92	27	99	325	70			7	95			95
19	90	179	99	173	80			7	94			94
20	90	269	98	83	84			7	94			94
21	92	352	102	6	98					1	97	97
22	91	351	102			7	63			1	63	63
23	90	358	101							1	97	97
24	91	357	101			1	88			1	88	88
25	90	359	101									101
26	92	359	103			48	32			1	34	34
27	91	408	101									101
28	92	6	95	352	66					50	63	63

Data for groups 13, 15, 16, and 17 show the moisture content at 1 year, after the specimens had been subjected to moist curing followed by air curing for periods of various length. As might be expected, the percentage of water remaining at the end of the drying period decreased as the period of air curing increased. This was true also for groups 14, 18, 19, and 20 insofar as the amount of moisture remaining at the end of the air curing period was concerned. However, these specimens were subjected to 7 days of water curing following the air curing period. For the limestone concrete, this final immersion in water resulted in final moisture contents approximately the same as before air curing began. However, the final moisture content of the Haydite concrete specimens varied with the length of the air curing period—the shorter this period the greater the water content at the end of the final 7 days of water curing.

Data for groups 10 and 21 show for both types of concrete that the moisture loss after 7 days in air curing was approximately the same for specimens that were cured initially for 1 year in the moist room as it was for specimens cured for 6 months. On the other hand, 7 days of oven curing at 170° F. for specimens previously cured for 6 months in the moist room reduced their water contents by 43 percent, as compared with a reduction of only 33 percent for specimens cured for 1 year (groups 11 and 22, respectively). Data for groups 23 and 24 show that for specimens cured initially 1 year in the moist room, drying in the oven for 1 day drove off three or four times as much water as drying for 1 day in air.

In order to determine how much water could be driven off at a temperature of 170° F., a group of both Haydite and limestone specimens were kept in the oven until they had reached constant weight. The results show (group 26) that a slightly greater percentage of

TABLE 6.—*Curing conditions and flexural strengths of concrete beams*

Group no.	Age at test	Curing				Coarse aggregate				Strength ratio, Haydite concrete to limestone concrete
		Initial moist	Oven	Air	Water	Haydite		Limestone		
						Modulus of rupture	Coefficient of variation	Modulus of rupture	Coefficient of variation	
	Days	*Days*	*Days*	*Days*	*Days*	*Lb. per sq. in.*	*Pct.*	*Lb. per sq. in.*	*Pct.*	*Pct.*
1	7	7				386	6.6	445	6.2	87
2	28	7		21		357	8.3	442	4.5	81
3	28	28				533	1.6	591	12.8	90
4	180	7		173		556	4.3	643	11.2	86
5	180	7		166	7	550	11.0	586	3.2	94
6	180	7		169	4	544	7.1	573	10.1	95
7	180	7		170	3	[1] 572	[1] 4.8	589	12.9	97
8	180	7		171	2	590	8.8	600	4.2	98
9	180	7		172	1	[1] 607	[1] 4.4	675	4.5	90
10	180	173		7		391	13.2	[1] 534	[1] 9.0	73
11	180	172	7	1		372	15.4	[1] 560	[1] 6.2	66
12	180	180				629	6.5	770	5.9	82
13	360	7		353		512	4.2	623	13.2	82
14	360	7		346	7	483	10.9	535	4.5	90
15	360	28		332		468	16.6	656	8.0	71
16	360	180		180		246	2.9	657	6.5	37
17	360	270		90		296	11.7	652	8.1	45
18	360	28		325	7	660	3.2	627	11.5	105
19	360	180		173	7	585	4.4	809	8.6	72
20	360	270		83	7	540	8.7	800	5.4	68
21	360	353		7		422	7.6	589	7.1	72
22	360	352	7	1		305	6.4	612	5.0	50
23	360	359		1		476	3.4	577	8.4	82
24	360	358	1	1		376	6.6	[1] 631	[1] 7.4	60
25	360	360				679	8.2	736	5.6	92
26	[2] 428	360	67	1		452	10.4			76
	[3] 409	360	48	1				591	14.9	
27	[2] 428	428				675	8.2			97
	[3] 409	409						695	7.8	
28	[2] 428	7		421		534	10.7			80
	[3] 409	7		402				668	5.8	
Average						493	7.7	624	7.8	79

[1] Values represent average of tests on 4 specimens, all other values represent average of tests on 5 specimens.
[2] Data for beams having Haydite aggregates.
[3] Data for beams having natural sand and crushed limestone aggregates.

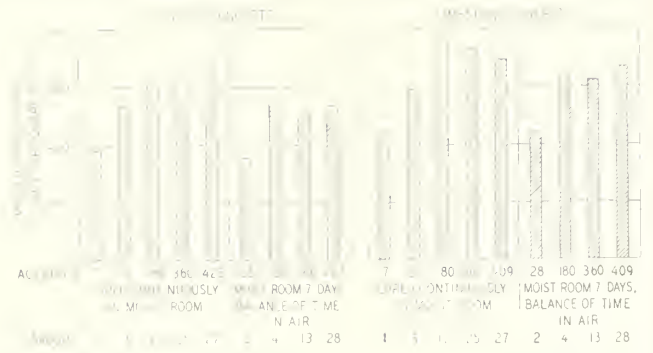

FIGURE 1.—RELATIONS BETWEEN AGE AND FLEXURAL STRENGTH OF HAYDITE AND LIMESTONE CONCRETE SPECIMENS.

water was lost by the Haydite concrete than by the limestone concrete, the percentages of water retained being 29 and 32, respectively. However, 67 days were required for the Haydite concrete to reach constant weight, as compared with 48 days for the limestone concrete.

Results of the absorption tests are of interest principally as indicating the effect of type of aggregate on rate of change in moisture content resulting from drying or absorption. In general, Haydite concrete both lost and gained moisture at a slower rate than did limestone concrete. In this connection it should be remembered that the concrete made with Haydite aggregates contained 7.2 sacks of cement per cubic yard, while concrete made with natural aggregates contained but 5.7 sacks per cubic yard.

NONUNIFORM MOISTURE DISTRIBUTION FOUND TO AFFECT FLEXURAL STRENGTH

The results of tests for flexural strength at various ages and for the several conditions of curing investigated are given in table 6 and are shown graphically in figures 1 to 6, inclusive. The relations between flexural strength and age under continuous moist and continuous air curing conditions for both types of aggregate are shown in figure 1. It will be observed that under moist curing the Haydite concrete increased in strength up to 360 days, and specimens tested at the age of 428 days gave practically the same results as those tested at 1 year.

For limestone concrete under moist curing there was a distinct decrease in strength at 360 days as compared with strength at 180 days, with still further retrogression at 409 days. The retrogression in strength after 6 months for the limestone concrete is quite surprising in view of the fact that standard moist curing conditions obtained throughout the curing period.

In the case of specimens cured in air after 7 days of initial moist curing, we find little change in strength after 180 days for both types of aggregate. The maximum strength attained by the Haydite concrete specimens cured in air was about the same as the 28-day strength after moist curing. For the limestone concrete specimens cured in air the maximum strength was slightly higher than the 28-day strength after moist curing. Also, the 28-day strengths (after curing in air for 21 days) were about the same as the strengths of the 7-day, moist-cured specimens. In this case the normal gain in strength between 7 and 28 days was probably neutralized by the fact that specimens air dried for 21 days were only partially dry when tested; that is, while the outer shell of the specimen was fairly dry the core

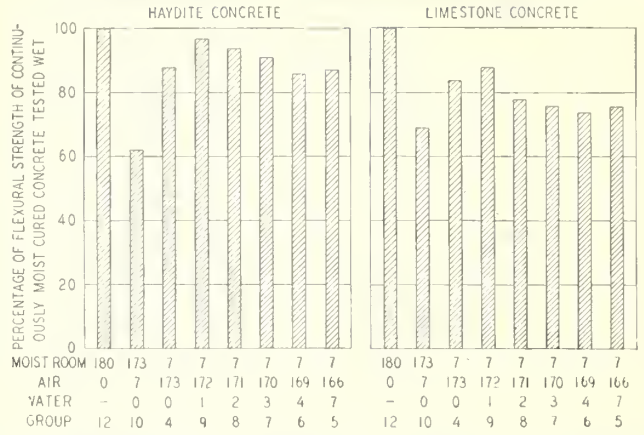
HAYDITE CONCRETE
LIMESTONE CONCRETE
PERCENTAGE OF FLEXURAL STRENGTH OF CONTINUOUSLY MOIST CURED CONCRETE TESTED WET
100
80
60
40
20
0
MOIST ROOM 180 173 7 7 7 7 7 7
AIR 0 7 173 172 171 170 169 166
WATER – 0 0 1 2 3 4 7
GROUP 12 10 4 9 8 7 6 5
180 173 7 7 7 7 7 7
0 7 173 172 171 170 169 166
– 0 0 1 2 3 4 7
12 10 4 9 8 7 6 5

FIGURE 2.—EFFECT OF VARIATIONS IN PERIOD OF RESATURATION ON FLEXURAL STRENGTH OF CONCRETE SPECIMENS AFTER CURING 180 DAYS AS INDICATED.

was still wet. The drying shell produced tensile stresses in the extreme fibers of the beam.

These initial stresses of course tended to reduce the load required to produce failure in bending, resulting in an observed modulus of rupture considerably lower than would have been obtained if the moisture content of the specimens when tested had been the same throughout. The fact that the strengths after 180 days in air curing increased materially probably resulted from the fact that during this period the specimens had an opportunity to dry uniformly, thus relieving these moisture stresses. That this so-called "shell" effect is of great importance in influencing the observed modulus of rupture will be evident from consideration of the results for specimens cured for 6 months and for 1 year, a discussion of which follows.

The results of flexure tests on specimens cured for 180 days under various curing and moisture conditions are shown in figure 2. Values in all cases have been expressed as percentages of the strength developed under continuous moist curing. It will be noted that specimens taken from the moist room at 173 days and cured in air for 7 days before testing attained only about two-thirds of the strength of specimens tested immediately upon removal from the moist room (groups 12 and 10). This decrease no doubt resulted from the partial drying of the specimens during the final 7-day period, as explained above. The effect was somewhat more marked in the Haydite concrete, the decrease in strength being 38 percent as compared with 31 percent for the limestone concrete.

The effect of partial drying is further illustrated by comparing the strengths of specimens cured in the moist room for 173 days, followed by 7 days in air, with those cured for 7 days in the moist room, followed by 173 days in air (groups 10 and 4). The air-cured specimens had much higher strengths, probably as a result of the absence of internal shrinkage stress in the concrete.

The effect of curing in water from 1 to 7 days just prior to testing the air-cured specimens is also shown in figure 2 (groups 5 to 9, inclusive). It will be observed that immersing the specimens for 1 day increased the strength, but that further immersion tended to lower the strength. This applied to both the Haydite and the limestone concretes. Here we have a condition where the dry specimen was absorbing water from the outside, resulting in an effect just the reverse of that previously described in which the wet specimen was

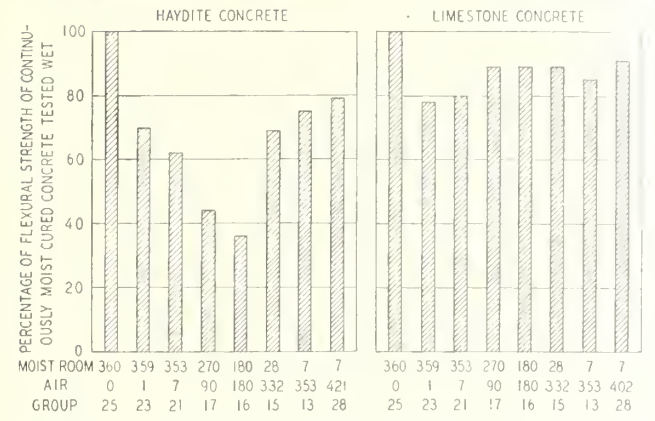
HAYDITE CONCRETE
LIMESTONE CONCRETE
PERCENTAGE OF FLEXURAL STRENGTH OF CONTINUOUSLY MOIST CURED CONCRETE TESTED WET
100
80
60
40
20
0
MOIST ROOM 360 359 353 270 180 28 7 7 360 359 353 270 180 28 7 7
AIR 0 1 7 90 180 332 353 421 0 1 7 90 180 332 353 402
GROUP 25 23 21 17 16 15 13 28 25 23 21 17 16 15 13 28

FIGURE 3.—EFFECT OF VARIATIONS IN PERIODS OF MOIST ROOM AND AIR CURING ON FLEXURAL STRENGTH OF CONCRETE SPECIMENS AFTER CURING 1 YEAR AS INDICATED.

drying. In this case the effect of moisture in the outer shell was to produce compressive stresses in the extreme fibers of the beam, thereby raising, instead of lowering, the observed modulus of rupture. That this is true is evidenced by the fact that specimens immersed for only 1 day in water had higher strengths than the specimens tested dry, but the strengths decreased as the period of immersion increased.

The strength of Haydite concrete after 4 days of immersion was practically the same as that of the air-dry specimens, indicating that moisture equilibrium had been established. For limestone concrete, the strengths after 2 days of immersion and longer were all somewhat lower than those of the air-dried specimens. For immersion periods of 3 days and longer there was little difference in strength. The results indicate that when curing has been in air, a 24-hour water immersion period prior to testing may not give true results because of incomplete moisture penetration.

EFFECT OF PERIOD AND TYPE OF CURING ON FLEXURAL STRENGTH STUDIED

The effect of variations in the relative amounts of moist and air curing on the flexural strength of specimens tested at the age of 1 year is shown in figure 3. It will be observed that, for Haydite concrete, increasing the period of air curing prior to test caused a progressive reduction in flexural strength up to and including 180 days of air curing. For this condition, the specimens developed only 36 percent of the strength of the continuously moist-cured concrete. However, further increasing the period of air curing up to 353 days resulted in increasing the strength progressively. An increase in the drying period up to 421 days still further increased the strength. It is believed, however, that the increase was caused more by the change in moisture condition than by the additional curing that the specimens received between 360 and 428 days.

The very peculiar behavior of the Haydite concrete here indicated is believed also to be caused by internal moisture stresses, as discussed above. There seems to be no other explanation that will cover the facts. In this connection it is interesting to note that, although the limestone concrete exhibited the same tendency, the maximum decrease in strength was not as great, and also that it occurred after only 1 day of drying. The moisture distribution appeared to be equalized, at least insofar as effect on internal stress was concerned, somewhere between 7 and 90 days of air curing, whereas in

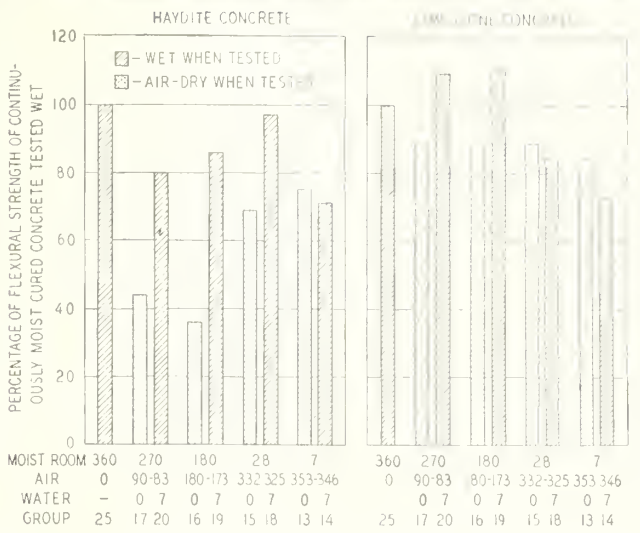

FIGURE 4.—EFFECT ON FLEXURAL STRENGTH OF RESATURATION FOR 7 DAYS PRIOR TO TESTING FOR CONCRETE SPECIMENS AFTER CURING 360 DAYS AS INDICATED.

the Haydite concrete it did not occur until certainly well beyond the 180-day period.

By comparing the results of the strength tests with the corresponding moisture losses at the time of test as shown in table 5 it will be noted also that the measured moisture loss appeared to bear no relation to variation in strength caused by moisture. For instance, groups 13, 15, 16, and 17 for limestone concrete showed practically the same strength although the percentage of total moisture retained varied from 67 to 85.

In order to study the effect of resaturation on the strength of the concrete cured for various periods in air, certain of the specimens cured as indicated in figure 3 were removed from air curing and immersed in water for 7 days prior to testing. The results are shown in figure 4, together with values for the corresponding air-dried specimens. In studying these results it should be noted that, insofar as curing is concerned, each pair of values (for instance, groups 16 and 19) represent tests on specimens subjected, except for the final 7-day period, to identical curing conditions. Therefore, the variations noted must have been caused by some other factor, presumably the shell effect previously discussed.

FLEXURAL STRENGTH OF HAYDITE CONCRETE FOUND MOST SUSCEPTIBLE TO MOISTURE CHANGE

It is difficult to explain satisfactorily all of the trends observed in figure 4. This applies particularly to the results obtained on the specimens immersed in water for 7 days prior to test. This period would certainly seem sufficient to eliminate the shell effect. However, if we assume this to be true, then variations in strength of the resaturated specimens must be caused by variations in curing only. This would not explain the relative strengths shown by the Haydite concrete in groups 18, 19, and 20. It would certainly be reasonable to expect the strength to decrease with decrease in the length of the initial moist curing period, whereas, with the single exception of the specimens cured for 7 days in the moist room (group 14), just the reverse is true.

In the limestone concrete the relative values are as would be expected, except for the high strengths shown for groups 19 and 20. It is possible that in these groups the reabsorption of water actually produced compression

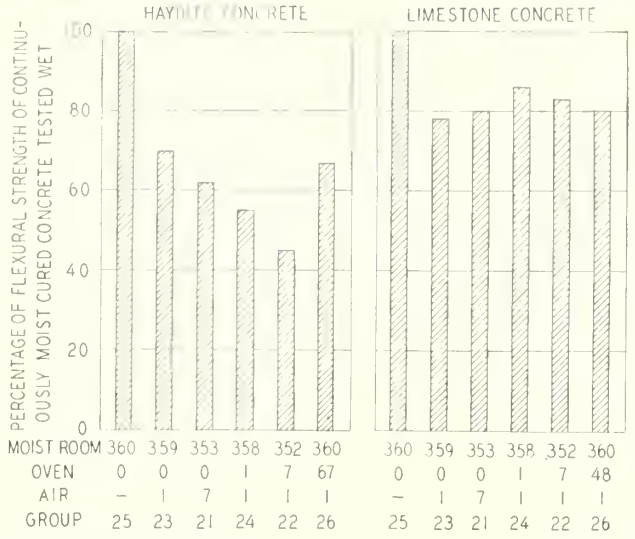
HAYDITE CONCRETE
LIMESTONE CONCRETE
PERCENTAGE OF FLEXURAL STRENGTH OF CONTINUOUSLY MOIST CURED CONCRETE TESTED WET
100
80
60
40
20
0
MOIST ROOM 360 359 353 358 352 360 360 359 353 358 352 360
OVEN 0 0 0 1 7 67 0 0 0 1 7 48
AIR – 1 7 1 1 1 – 1 7 1 1 1
GROUP 25 23 21 24 22 26 25 23 21 24 22 26

FIGURE 5.—EFFECT OF DRYING PRIOR TO TEST ON FLEXURAL STRENGTH OF CONCRETE SPECIMENS AFTER CURING APPROXIMATELY 1 YEAR IN MOIST ROOM.

in the extreme fibers which, combined with the relatively long inital period of moist air curing, produced the exceptionally high strengths observed. In any event, it is obvious that, in the determination of the flexural strength of concrete, great care should be exercised to insure that the specimens are truly in equilibrium, insofar as internal moisture stresses are concerned, when they are tested.

In the discussion so far presented the specimens cured in air were partially dried by storing in an unheated room adjacent to the laboratory. When the outside temperature went below freezing, a window leading to the laboratory was opened so that with the exception of short periods the temperature in the curing room was kept above freezing.

Tests were made to determine the effect of rapid drying and also to determine what effect drying to constant weight would have on the flexural strength. A number of specimens were dried in an electric oven at a temperature of approximately 170° F. for periods of 1 and 7 days. Others were dried to constant weight, the time required varying for the individual specimens in each group. The 48 days noted for the limestone concrete and the 67 days noted for the Haydite concrete represent the average of five separate tests in each case. After removal from the oven the specimens were allowed to cool for 1 day in the laboratory before being tested. The results are shown in figure 5.

Referring first to the results for Haydite concrete, it will be observed that the strengths progressively decreased as the amount of drying increased up to 7 days of oven curing. Haydite specimens subjected to this treatment developed only 45 percent of the strength of moist-cured specimens of the same age, as compared with 83 percent for the limestone specimens. It will be noted also that, with the limestone concrete, the greatest decrease in strength occurred after 1 day of air drying, as against 7 days of oven and 1 day of air drying for the Haydite concrete. These results again point to the greater susceptibility of the Haydite concrete to moisture change and show also the effect of the slower drying action in concrete containing Haydite.

RECOMMENDATION MADE THAT FLEXURE SPECIMENS BE IMMERSED IN WATER FOR 48 HOURS BEFORE TESTING

Comparison of the flexural strengths of the Haydite and the limestone concrete specimens subjected to similar curing conditions shows that the average for all Haydite specimens tested wet was about 90 percent of the strength of the limestone concrete. On the other hand, the average strength of all Haydite specimens tested in various degrees of dryness was only about 70 percent of the strength of the limestone concrete cured and tested in like manner. This indicates that the strength of the Haydite concrete was affected to a greater degree by air curing than was the limestone concrete.

The degree to which the Haydite specimens were adversely affected varied over a wide range. For instance, the strength of Haydite specimens cured for 173 days in air after 7 days of moist curing was 86 percent of that of limestone specimens, or nearly equal to the 90 percent for the continuously moist-cured specimens. On the other hand, the Haydite specimens cured for 180 days in air after 180 days of moist curing had but 37 percent of the strength of limestone concrete similarly cured. It is apparent, therefore, that the time at which dry curing began had a marked effect on the ratio of the strength of the Haydite to the strength of the limestone concrete.

The previous discussion has been confined to the effect of the several variables on the flexural strength of the concrete tested. Additional tests were made to determine the effect on crushing strength of air curing as compared with moist curing. Tests were made at 7, 28, 180, and 360 days, on specimens cured the entire time in the moist room and on specimens cured in air after an initial moist curing of 7 days. No tests were made on specimens cured dry and tested wet or on specimens cured wet and tested dry as was done with the beam specimens. In other words, all of the compression specimens were probably in a stabilized moisture condition when tested.

The results for the Haydite concrete (fig. 6) indicate a progressive increase in strength for moist curing up to 180 days, with a small increase over the 180-day strength at 360 days. The same general trend is found for the specimens cured in air; that is, the strength at 360 days was approximately the same as that at 180 days. At each curing period the air-cured specimens had lower strengths than the moist-cured specimens, the difference becoming greater as the age at test increased. The results are in agreement with those found in the flexure tests; that is, the specimens cured in air and tested dry had lower strengths than those cured in moist air and tested wet.

The right-hand panel of figure 6 gives similar results for the limestone concrete. The results obtained are in general agreement with those found for the Haydite concrete; therefore they will not be discussed in detail. They are also in agreement with the flexure test results where the specimens were subjected to like curing treatments.

These tests indicate that the flexural strength of concrete containing aggregates of the type represented by Haydite is affected considerably more by variations in curing and moisture content at time of test than is concrete containing naturally occurring aggregates of average quality.

(Continued on p. 61)

THE OLIENSIS SPOT TEST IMPROVED

Reported by R. H. LEWIS, Associate Chemist, and J. Y. WELBORN, Junior Highway Engineer, Division of Tests, Bureau of Public Roads

SINCE the publication in 1933 of Mr. Oliensis' paper "A Qualitative Test for Determining the Degree of Heterogeneity of Asphalts",[1] the Bureau of Public Roads has devoted considerable time to the study of this test, which has proven to be a very useful one. The results of these studies corroborate the author's conclusions with respect to the scope and limitations of the test.

In tests by the Bureau on a large number of asphaltic materials from all producing centers, heterogeneity is found to be far more prevalent among the more fluid materials than among the refined, semisolid, paving asphalts, and the quantitative method recently developed by Mr. Oliensis should be of great value in comparing the degree of heterogeneity of various asphaltic materials. The paper[2] describing this new method of test has been studied and the data obtained are presented in this report.

Clifford Richardson has stated: [3]

> In residual pitches, at times some of the bitumen is found which is insoluble in cold carbon tetrachloride, and this is evidently due to the severe treatment which the material has suffered in the course of its production at very high temperatures. A determination of the amount is only valuable as an indication of the care which has been used in the preparation of such pitches.

For years, therefore, asphalt specifications, have required a high solubility in carbon tetrachloride as a protection against overheating in the refining process.

However, examination of fluid and semisolid asphalts from numerous sources has shown that many of the products that have been subjected to temperatures much higher than those used in normal steam refining have relatively high solubility in carbon tetrachloride. Many materials that are definitely cracking-coil residues produce a positive stain with both naphtha and xylene, while the majority of those that apparently have been inadvertently overheated in a steam or vacuum process, and some blends of cracked and uncracked residuals, give a positive stain with naphtha and a negative stain with xylene. In spite of their high solubility in carbon tetrachloride, these asphaltic materials are heterogeneous. It is apparent, therefore, as has been pointed out by Mr. Oliensis, that a high solubility in carbon tetrachloride does not indicate definitely that the material has not been overheated in the refining process.

The Bureau is now engaged in a detailed investigation of commercial grades of semisolid asphalts from the refineries of the major producers. This study involves among other things the exposure of the asphalts in films one-eighth inch thick to the action of sun and light for 15 weeks (approximately 900 sunlight hours).

Thirty-nine samples of 85–100 penetration asphalt have been studied, of which 16 samples gave negative stains both before and after exposure. Nine gave negative stains before exposure and showed varying degrees of heterogeneity after exposure. Twelve gave positive stains before and after exposure; and two materials that showed slight heterogeneity before exposure produced negative stains after 15 weeks of exposure. Perhaps these two are examples of the fugitive type which has been observed by Mr. Oliensis.

For the 23 samples that gave positive stains either before or after exposure, table 1 gives the base petroleum and the refining process as reported by the manufacturers. Results of the Oliensis spot test before and after exposure, the gilsonite and xylene equivalents before exposure, and the xylene equivalent after exposure are also listed.

METHOD OF DETERMINING GILSONITE AND XYLENE EQUIVALENTS DESCRIBED

In considering the data in table 1, and later in table 2, the heterogeneity of the materials is expressed quantitatively by limits representing the highest percentage of gilsonite or xylene that would produce a positive stain and the lowest percentage of gilsonite or xylene that would produce a negative stain. The true xylene or gilsonite equivalent (the exact percentage of homogeniser necessary to produce a negative stain) is within these limits. Thus, a xylene equivalent of 12–16 means that with 12 percent or less of xylene in the standard naphtha the material developed a positive stain and that with 16 percent or more of xylene, a negative stain was obtained. A gilsonite equivalent of 50–60 means that when 50 percent of gilsonite was blended with the material under test and the blend tested with standard naphtha a positive stain was produced; when the blend of gilsonite and asphaltic material contained 60 percent of gilsonite a negative stain was obtained. The term "xylene insoluble" indicates that a positive stain was obtained with 100 percent of xylene and the material is beyond the range of the test, i. e., the heterogeneity of the material is indeterminable with this solvent.

Table 1 shows that only 2 of the 39 materials produced a positive stain with xylene, either before or after exposure. Eight of the 14 original materials that were heterogeneous had xylene equivalents of 8 or less, and gilsonite equivalents of 10 or less. Of the heterogeneous exposure residues of the nine materials that were originally homogeneous, seven had xylene equivalents of eight or less. For those materials that were heterogeneous originally, the increase in xylene equivalent in the exposed residues was quite variable.

Samples 7 and 8 are of special interest. Both were produced from the same Venezuelan crude, the first by the continuous vacuum steam distillation and the latter by batch steam distillation. After exposure sample 8, which gave a positive stain initially, had a higher xylene equivalent than sample 7, which gave a negative stain initially.

In table 1 of Mr. Oliensis' report,[2] he shows that the xylene equivalent for sample 13, representing a wax-bearing residual, crude K, was greater than the gilsonite equivalent. Sample 23 of the materials tested by the Bureau was a wax-bearing residual, produced from a crude that tested positive in the spot test. Tests made on other semisolid products from this same refinery and on residues reduced in the laboratory with a high

[1] Proceedings, American Society for Testing Materials, vol. 33, pt. II.
[2] A Further Study of the Heterogeneity of Asphalt—A Quantitative Method, by G. L. Oliensis. Proceedings, American Society for Testing Materials, vol. 36, pt. II.
[3] The Modern Asphalt Pavement, by Clifford Richardson. John Wiley & Sons, New York, 1905, 1 ed., p. 120.

TABLE 1.—*Oliensis spot test results and gilsonite and xylene equivalents for samples of 85–100 penetration asphalt*

Sample no.	Base petroleum	Refining process	Tests before exposure			Tests after exposure for 15 weeks in ⅛-inch film	
			Standard naphtha stain	Gilsonite equivalent	Xylene equivalent	Standard naphtha stain	Xylene equivalent
1	California	Steam distillation	Negative			Positive	0–2
2	do	Vacuum distillation	do			do	0–2
3	do	Steam distillation	do			do	12–16
4	do	Vacuum distillation	Positive	0–1	0–2	do	12–16
5	Mexican		do	50–60	24–28	do	28–32
6	Venezuelan	Steam distillation	do	5–8	2–4	do	12–16
7	do	Continuous vacuum steam distillation	Negative			do	12–16
8	do	Batch steam distillation	Positive	5–10	2–4	do	20–24
9	do	Steam distillation	do	5–10	2–4	do	12–16
10	do	Vacuum distillation	Negative			do	0–2
11	do	Steam distillation	do			do	0–2
12	Smackover	Vacuum distillation	Positive	0–1	0–2	Negative	
13	Oklahoma		do	30–35	44–48	Positive	72–76
14	do		do	2–4	2–4	do	4–8
15	do		do	5–10	4–8	do	8–12
16	do		do	60–80	(1)	do	(1)
17	Blend of Mexican and Oklahoma	Steam distillation	Negative			do	4–8
18	Kansas	Winkler-Kock shell still	Positive	80–90	(1)	do	(1)
19		Steam distillation	do	15–25	12–16	do	32–36
20		do	do	0–2	0–2	Negative	
21	Blend of Mexican and domestic Gulf coast crudes.		Negative			Positive	0–2
22	do		do			do	2–4
23	Texas		Positive	35–40	16–20	do	36–40

[1] Xylene insoluble.

steam ratio from the crude oils used as base petroleum by this refinery, showed that they required approximately 35 percent of gilsonite to produce a negative stain. Although the other samples from this source were not tested for xylene equivalent, it is noted that the xylene equivalent for sample 23 was considerably lower than the gilsonite equivalent. This is not in agreement with the conclusion by Mr. Oliensis that gilsonite is more efficient than xylene in correcting heterogeneity caused by waxy bodies.

The spot test results for these representative asphalts show that 23 out of 39 samples, or about 60 percent, were homogeneous, and that 16, or about 40 percent, remained homogeneous after 15 weeks of exposure. All of the homogeneous materials that became heterogeneous upon exposure had xylene equivalents of 16 or less. Under the climatic conditions existing during the winter months at Madison, Ill., it is possible that heterogeneity might not have developed in these samples.

NEW METHOD USEFUL IN TESTING BOTH ORIGINAL MATERIALS AND RESIDUES FROM EXPOSURE

Under the same summer conditions to which the 85–100 penetration asphalts were subjected, all fluid materials that were studied in this investigation developed heterogeneous residues within 5 weeks. In a previous report covering a study of the spot test on liquid asphalts, mentioned in Mr Oliensis' paper, the following statement was made: [4]

> Since the results of the tests were based upon the appearance of the stain as interpreted by the observer, it is difficult if not impossible to distinguish between border-line materials or to express clearly the apparent degree of heterogeneity that may be indicated by the varying degrees of nonuniformity in the stain. The classification given should be understood to mean that, in the judgment of the observers, the materials and their residues gave stains that appeared either entirely uniform throughout or were only slightly nonuniform, having a slightly darker, more pronounced center, or else they had a definite dark to black center surrounded by a uniformly lighter colored stain, and were consequently classified as homogeneous, slightly heterogeneous, and heterogeneous respectively.

[4] Further Studies of Liquid Asphaltic Road Materials, by R. H. Lewis and W. O'B. Hillman. PUBLIC ROADS, vol. 16, no. 6, August 1935, p. 108.

The use of the xylene equivalent gives promise of overcoming past difficulties in the proper interpretation of the spot test results and appears to offer a ready means of determining the comparative degree of heterogeneity and the rate of its development in materials exposed to weathering.

In a report presented at the January 1936 meeting of the Association of Asphalt Paving Technologists [5] the behavior of five materials—three steam-distilled residuals (Mexican, midcontinent, and California), and two cracking coil residuals under exposure conditions—was described. The results of the spot tests, determined on these materials after exposure in films 1/8, 1/16, and 1/32 inch thick for 5, 10, and 15 weeks, may help to show the possibilities of the quantitative method of determining heterogeneity suggested by Mr. Oliensis.

The two cracking coil residues gave positive spots with xylene, and the development of heterogeneity in these materials under exposure could only be detected by the steady development of organic matter insoluble in CS_2 and CCl_4. The other three materials and their distillation residues gave negative stains. The xylene equivalents of the residues after various periods of exposure are shown in table 2.

It will be seen that, at each test period, the thinner the film thickness the greater the xylene equivalent. For each thickness of film, except the ⅛- and 1/16-inch films of the Mexican residual, the xylene equivalent increased with the time of exposure. The reason that the ⅛- and 1/16-inch films of the Mexican residual after 10 weeks of exposure had lower xylene equivalents than the corresponding films after 5 weeks of exposure is not known.

Data on the solubilities of some of these residues are of interest (table 3). It will be seen that the 1/32-inch film of sample 2 after 15 weeks of exposure had more matter insoluble in CS_2 than two of the other residues, and more matter insoluble in CCl_4 than three of the other residues. However, it was readily dispersed in 68–72 percent xylene-naphtha solution, while the other samples all contained material insoluble in 100 percent xylene. This would seem to indicate a difference in the

[5] A Report on the Weather-Resistant Properties of Certain Slow-Curing Liquid Asphaltic Materials. Proceedings of the Association of Asphalt Paving Technologists, Jan. 23, 1936.

TABLE 2.—*Xylene equivalents of residues of slow-curing liquid asphaltic materials (grade SC—2) after exposure*

Samples tested after exposure for—	Xylene equivalent of residues of—								
	Sample 1, Mexican residual. Film thickness of—			Sample 2, midcontinent residual. Film thickness of—			Sample 3, California residual. Film thickness of—		
	⅛-inch	1/16-inch	1/32-inch	⅛-inch	1/16-inch	1/32-inch	⅛-inch	1/16-inch	1/32-inch
5 weeks	4–8	8–12	12–16	0–2	28–32	48–52	24–28	36–40	48–52
10 weeks	0–2	4–8	(1)	16–20	36–40	60–64	32–36	44–48	64–68
15 weeks	8–16	(1)	(1)	36–40	56–60	68–72	36–40	48–52	(1)

[1] Xylene insoluble.

character of the insoluble materials in the several residues that affects their dispersion in the solvents used for determining heterogeneity.

The stain obtained in the original Oliensis spot test indicates only that the material is homogeneous or heterogeneous. The degree of heterogeneity is not determinable by this test and, as indicated in table 1, it automatically classes materials that have been only slightly overheated with products that have been subjected to excessive heat. This quantitative method, therefore, should prove extremely valuable in future studies of asphaltic materials.

TABLE 3.—*Comparison of solubilities of exposed samples with xylene equivalents*

Sample no.	Time of exposure	Film thickness	Total organic matter insoluble in CS_2	Total organic matter insoluble in CCl_4	Xylene equivalent
	Weeks	*Inch*	*Percent*	*Percent*	
1	10	1/32	1.88	1.98	Insoluble.
1	15	1/16	1.42	1.67	Do.
1	15	1/32	2.88	3.22	Do.
2	15	1/32	1.65	2.06	68–72.
3	15	1/32	1.21	1.40	Insoluble.

(Continued from p. 58)

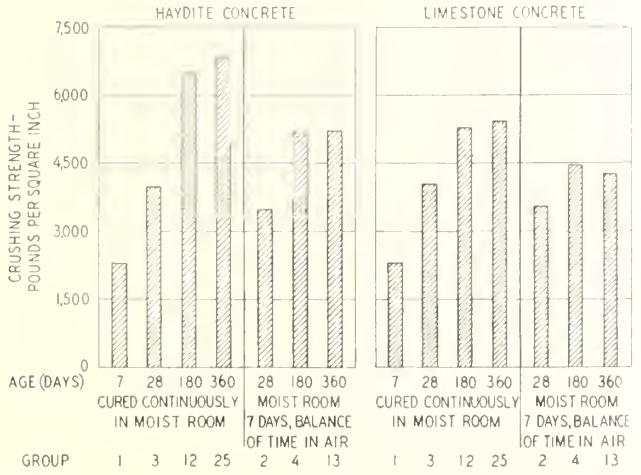

FIGURE 6.—RELATIONS BETWEEN AGE AND CRUSHING STRENGTH OF CONCRETE SPECIMENS CURED IN VARIOUS WAYS.

The wide variations in flexural strength of the Haydite concrete appear to result primarily from its tendency to absorb and to give off moisture at a slower rate than concrete containing normal aggregates, thus delaying the even distribution of moisture which is necessary in order to eliminate internal stresses.

The results emphasize the importance of controlling testing procedure so as to insure that specimens under test are free from stresses induced by an uneven distribution of moisture. They indicate that, where curing has been in air, immersion in water prior to test for more than the customary 24 hours may be necessary and that, when the specimens have been subjected to continuous moist curing until test, care should be exercised to prevent any drying prior to test.

Drying to constant weight in order to eliminate moisture stresses is not practicable because of the length of time required. For this reason it is recommended that all flexure testing be performed on saturated specimens, and that in cases where specimens have been subjected to drying prior to test they be immersed in water for at least 48 hours immediately before testing.

SLIPPERINESS OF ROAD SURFACES STUDIED BY ENGLISH INVESTIGATORS

Apparatus for accurately measuring the slipperiness of various road surfaces has been developed at the Road Research Laboratory of the Department of Scientific and Industrial Research in England. The machine used consists of a motorcycle and sidecar in which the sidecar wheel can be set at an angle to the direction of travel.

The horizontal reaction of the road, developed by the wheel because of its position, and the load on the wheel, are measured. From data obtained with instruments carried in the sidecar, the ratio of these forces is obtained and expressed as a coefficient which is high for nonskid surfaces and low for slippery ones. The construction and operation of the motorcycle and sidecar and of the measuring apparatus are described in a bulletin [1] recently issued jointly by the Department of Scientific and Industrial Research and the Ministry of Transport. The apparatus, which has been in regular use for several years, is estimated to cost £475 (approximately $2,350).

A bulletin [2] reporting results of the first of a series of studies of road friction has also been published recently. This bulletin summarizes the results of numerous measurements made on various types of road surfaces under different climatic conditions and at different speeds. Deductions and conclusions are drawn from the results obtained.

The skidding coefficient was found to be high for dry road surfaces free from loose material, and such surfaces can be considered as nonskid. On wet surfaces, the coefficient decreases as the speed increases. A value of 0.5 at 30 miles per hour may be regarded as reasonably safe; a value of 0.2 indicates that the surface affords insufficient resistance to skidding. Values of 0.2 or less are found most frequently on city streets.

The most slippery condition of a road is probably during a silver thaw, when the coefficient is as low as 0.1 at all speeds. Measurements taken on a snow-covered road showed values ranging from 0.5 to 0.2, the latter being found after the snow had become packed. The values for snow-covered roads were no lower than those found on some smooth, wet surfaces, but on snow-covered roads the lower values may persist down to speeds of 5 miles per hour.

On concrete surfaces the coefficient varied but little at different seasons of the year. Practically all other surfaces, when wet, were more slippery in summer than in winter. Tests made on wood- and rubber-block pavements showed them to have low coefficients.

The apparatus is now being improved to enable tests to be made at speeds higher than 30 miles per hour.

[1] Road Research Bulletin No. 1, Measurement of the Non-Skid Properties of Road Surfaces, H. M. Stationery Office, 9d net.

[2] Road Research Technical Paper No. 1, Studies in Road Friction. 1. Road Surface Resistance to Skidding. Published by H. M. Stationery Office, 1s. 6d. net.

MOTOR-FUEL CONSUMPTION, 1936

[Compiled for calendar year from reports of State authorities]

State	Tax rate per gallon— On Jan. 1	Tax rate per gallon— On Dec. 31	Date of rate change	Net total consumption in State [1]	Amounts exempted from payment of tax [2]: Federal and other public use [3]	Nonhighway use [4]	Allowance for evaporation and other losses	Total	Gross amount assessed for taxation	Amount subject to refund of entire tax	Net amount taxed	Amount taxed at full rate	Amount taxed at reduced rates: Rate per gallon	Amount	Approximate amount taxed for highway use [5]: 1936	1935	Percentage change
	Cents	*Cents*		*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*Cents*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	
Alabama	6	6		199,040					199,040		199,040	199,040			*199,040	*172,474	15.4
Arizona	5	5		96,543	5,475			5,475	91,068	11,559	79,509	79,509			79,509	67,323	18.1
Arkansas	6½	6½		154,208	4,472		1,460	5,932	148,276		148,276	133,884	(6)	14,392	*148,276	*131,784	12.5
California	3	3		1,651,837	27,824		16,240	44,064	1,607,773	147,780	1,459,993	1,459,993			1,459,993	1,340,137	8.9
Colorado	4	4		208,897	[7] 9,188	615	4,145	13,948	194,949	22,288	172,661	172,661			172,661	152,324	13.3
Connecticut	3	3		299,405	3,024		2,957	5,981	293,424	6,206	287,218	287,218			287,218	263,781	8.9
Delaware	4	4		50,766	[8] 1,024			1,024	49,742	2,675	47,067	47,067			47,067	42,948	9.6
Florida	7	7		360,192	7,642	1,993	2,478	12,113	288,079		288,079	288,079			*288,079	*256,609	12.3
Georgia	6	6		303,968	3,809		1,486	5,295	298,673		298,673	298,673			*298,673	*264,617	12.9
Idaho	5	5		87,848	4,331	478		4,809	83,039	8,135	74,904	74,701	2½	[9] 203	74,701	63,743	17.2
Illinois	3	3		1,191,915					1,191,915	68,583	1,123,332	1,123,332			1,123,332	1,015,019	10.7
Indiana	4	4		564,073	2,364			2,364	561,709	35,800	525,909	525,909			525,909	472,010	11.4
Iowa	3	3		460,298					460,298	48,754	411,544	411,544			411,544	386,489	6.5
Kansas	3	3		450,331		[10]124,886	9,007	133,893	316,438		316,438	316,438			316,438	295,308	7.2
Kentucky	5	5		228,333					228,333		228,333	228,333			*228,333	*201,324	13.4
Louisiana	5	7	July 28	223,093	3,853		6,693	10,546	212,547		212,547	[11] 212,547			*212,547	*186,201	14.1
Maine	4	4		134,521	1,412			1,412	133,109		133,109	127,513	1	[12] 5,596	127,513	114,532	11.3
Maryland	4	4		245,231	3,731			3,731	241,500	14,964	226,536	224,319	3	[13] 2,217	226,536	204,850	10.6
Massachusetts	3	3		654,309					654,309	29,379	624,930	624,930			624,930	584,233	7.0
Michigan	3	3		995,581	4,971	49,925	28,027	82,923	912,658	39,682	872,976	872,976		[14]	[15] 871,542	767,987	13.5
Minnesota	3	3		483,844	3,768	1,881	15,510	21,159	462,685	58,259	404,426	404,426			404,426	374,701	7.9
Mississippi	6	6		169,945	4,496		3,305	7,801	162,144		162,144	151,880	1	[16] 10,264	151,880	123,291	23.2
Missouri	2	2		567,750					567,750	49,539	548,211	548,211			548,211	498,350	10.0
Montana	5	5		111,743	3,813		2,116	5,929	105,814	15,367	90,447	90,447			90,447	77,393	16.9
Nebraska	5	5		231,581	1,274		7,085	8,359	223,222		223,222	223,222			*223,222	*219,165	1.8
Nevada	4	4		32,839	1,633	1,189	601	3,423	29,416	1,996	27,420	27,420			27,420	24,046	14.0
New Hampshire	4	4		80,898					80,898	2,150	78,748	78,748			78,748	71,992	9.4
New Jersey	3	3		723,024	[17] 12,193			12,193	710,831	52,977	657,854	657,854			657,854	594,432	10.7
New Mexico	5	5		81,858	4,622		1,165	5,787	76,071	6,525	69,546	69,546			69,546	57,987	19.9
New York	4	3	July 1	1,721,830	[18] 59,681	4,010		63,691	1,658,139	39,017	1,619,122	[19]1,619,122			1,619,122	1,495,863	8.2
North Carolina	6	6		350,380	3,750	638	2,315	6,703	343,677		343,677	335,097	1	[16] 8,580	335,097	305,579	9.7
North Dakota	3	3		103,697	877			877	102,820	25,799	77,021	77,021			77,021	78,877	−2.4
Ohio	4	4		[20] 1,201,445	4,216	29,893	33,985	68,094	1,133,351	9,998	1,123,353	1,078,255	1	[20] 45,098	1,078,255	965,240	11.7
Oklahoma	4	4		365,737	3,666		7,222	10,888	354,849	22,606	332,243	322,243			332,243	299,503	10.9
Oregon	5	5		216,578	4,663			4,663	211,915	25,050	186,865	186,092	1	[21] 773	186,092	160,434	16.0
Pennsylvania	4	4		1,283,280	6,055			6,055	1,277,225		1,277,225	1,277,225			*1,277,225	*1,171,439	9.0
Rhode Island	2	2		123,298	1,009			1,009	122,289	7,889	114,400	114,400			114,400	106,133	7.8
South Carolina	6	6		163,474	2,341			2,341	161,133		161,133	161,133			[22] *160,810	*143,014	12.4
South Dakota	4	4		116,043	1,970		4,563	6,533	109,510		109,510	98,447	2	[23] 11,063	98,447	[24] 96,531	2.0
Tennessee	7	7		266,315	[25] 11,122		2,528	13,650	252,665		252,665	252,665			*252,665	*216,386	16.8
Texas	4	4		1,108,063	12,453		10,976	23,409	1,084,654	125,697	958,957	958,957			958,957	835,942	14.7
Utah	4	4		83,358	2,758		2,417	5,175	78,183		78,183	78,183			[26] *77,912	*69,396	12.3
Vermont	4	4		60,026	2,329			2,329	57,697		57,697	57,697			*57,697	*51,388	12.3
Virginia	5	5		316,556					316,556	17,454	299,102	299,102			299,102	272,169	9.9
Washington	5	5		319,302	5,563			5,563	313,739	25,518	288,221	288,221			288,221	252,601	14.1
West Virginia	4	4		181,039	[27] 457			457	180,582	5,952	174,630	174,630			174,630	153,105	14.1
Wisconsin	4	4		503,969	3,161		12,521	15,682	488,287	35,778	452,509	452,509			452,509	405,909	11.5
Wyoming	4	4		58,044	1,748			1,748	56,296		56,296	56,296			[28] *55,482	*47,445	16.9
Dist. Columbia	2	2		126,837	[29] 4,299		293	4,592	122,245	620	121,625	121,625			121,625	111,983	8.6
Total	Weighted average rate 3.85 cents.			19,653,142	247,037	215,508	179,075	641,620	19,011,522	933,996	18,077,526	17,979,340		98,186	17,993,077	16,264,077	10.6

[1] Export sales and other amounts not representing consumption in State have been eliminated as far as possible. Exemptions and refunds in a few States include small amounts of such deductible items not reported separately. In cases where States failed to report amounts exempted from taxation the gross amount taxed is shown in this column.

[2] In cases where refunds were made for Federal and other public uses, or for losses, the amounts have been included with the exemptions listed below, rather than with the refund gallonage.

[3] Federal use except as otherwise noted.

[4] In Kansas exemptions rather than refunds are made for all nonhighway uses. Other States, as shown, make both exemptions and refunds. Florida exempts motor fuel used in aviation but not other nonhighway uses.

[5] 13 States do not provide for exemptions or refunds for nonhighway use. The amounts entered for these States, indicated by stars, include both highway and nonhighway uses.

[6] Within 300 feet of border tax rate is reduced to that of adjacent State. Gallons taxed at 2 cents, 2,739,000; at 4 cents, 11,534,000; at 5 cents, 119,000.

[7] Federal use, 5,415,000 gallons; State, county, and municipal use, 3,773,000 gallons.

[8] Federal use, 414,000 gallons; State use, 610,000 gallons.

[9] Motor fuel used in aviation.

[10] Includes Federal use.

[11] Includes 113,859,000 gallons taxed at 5 cents per gallon and 98,688,000 gallons taxed at 7 cents per gallon.

[12] 3 cents per gallon refunded on nonhighway uses.

[13] 1 cent per gallon refunded on motor fuel used in vehicles licensed to operate exclusively in cities.

[14] 1½ cents per gallon refunded on motor fuel used in interstate aviation. Amount not reported.

[15] Excludes 1,464,000 gallons of aviation fuel taxed at 3 cents per gallon.

[16] 5 cents per gallon refunded on nonhighway uses.

[17] Federal use, 3,335,000 gallons; State, county, and municipal use, 8,858,000 gallons.

[18] Federal use, 11,880,000 gallons; State, county, and municipal use, 47,801,000 gallons.

[19] Includes 705,520,000 gallons taxed at 4 cents per gallon and 913,602,000 gallons taxed at 3 cents per gallon.

[20] Does not include 69,610,000 gallons of liquid fuel (kerosene, fuel oil, etc.) taxed at 1 cent per gallon but not subject to the 3-cent tax on motor-vehicle fuel.

[21] 4 cents per gallon refunded on motor fuel used in aviation.

[22] Excludes 323,000 gallons of aviation fuel.

[23] 2 cents per gallon refunded on nonhighway uses.

[24] Revised figure.

[25] Federal use, 7,634,000 gallons; county and municipal use, 3,488,000 gallons.

[26] Excludes 271,000 gallons of aviation fuel.

[27] Public use not reported in full; amount includes 32,000 gallons Federal use and 425,000 gallons, State, county, and municipal use, on which refunds were paid.

[28] Excludes 814,000 gallons of aviation fuel.

[29] Includes both Federal and District Government use.

STATE MOTOR-FUEL TAX RECEIPTS, 1936

[Compiled for calendar year from reports of State authorities]

State	Tax rate per gallon— On Jan. 1	Tax rate per gallon— On Dec. 31	Date of rate change	Receipts from taxation of motor fuel: Gross receipts	Refunds paid	Net receipts	Other receipts in connection with motor-fuel tax [1]: Distributors' and dealers' licenses	Inspection fees [2]	Fines and penalties	Miscellaneous receipts [3]	Total	Net total receipts
	Cents	*Cents*		*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	6	6		11,754		11,754	*	49			49	11,803
Arizona	5	5		4,483	642	3,841	*		2		2	3,843
Arkansas	6½	6½		9,155		9,155		80			80	9,235
California	3	3		47,429	4,433	42,996	12				12	43,008
Colorado	4	4		7,855	1,022	6,833						6,833
Connecticut	3	3		8,967	185	8,782	53				53	8,835
Delaware	4	4		1,962	109	1,853	3		*		3	1,856
Florida	7	7		19,925		19,925	33	359			392	20,317
Georgia	6	6		17,493		17,493						17,493
Idaho	5	5		4,094	402	3,692	1		*	3	4	3,696
Illinois	3	3		35,398	1,940	33,458		359	2		361	33,819
Indiana	4	4		22,127	1,432	20,695	*	458		1	459	21,154
Iowa	3	3		13,585	1,389	12,196						12,196
Kansas	3	3		9,372		9,372	6	100		42	148	9,520
Kentucky	5	5		11,273		11,273			3	1	4	11,277
Louisiana	5	7	July 28	12,121		12,121		86	*		86	12,207
Maine	4	4		5,370	168	5,202	*					5,202
Maryland	4	4		9,542	621	8,921						8,921
Massachusetts	3	3		19,348	900	18,448						18,448
Michigan	3	3		26,925	1,190	25,735	3		1		4	25,739
Minnesota	3	3		13,994	1,861	12,133	1	194	1	*	196	12,329
Mississippi	6	6		9,577	515	9,062	*			[4]		9,062
Missouri	2	2		11,360	288	11,072		116			116	11,18
Montana	5	5		5,249	794	4,455						4,455
Nebraska	5	5		11,443	225	11,218		102			102	11,320
Nevada	4	4		1,166	86	1,080	*					1,080
New Hampshire	4	4		3,266	86	3,180				1	1	3,181
New Jersey	3	3		21,304	2,217	19,087	16		*	3	19	19,106
New Mexico	5	5		3,715	326	3,389	21				21	3,410
New York	4	3	July 1	57,221	1,587	55,634	59			16	75	55,709
North Carolina	6	6		20,423	429	19,994		962		5	967	20,961
North Dakota	3	3		3,088	843	2,245	*	56			56	2,301
Ohio	4	4		[5] 45,198	1,748	43,450	*					43,450
Oklahoma	4	4		14,094	883	13,211			5		5	13,216
Oregon	5	5		10,492	1,284	9,208			1	9	10	9,218
Pennsylvania	4	4		49,364		49,364			12	7	19	49,383
Rhode Island	2	2		2,396	170	2,226	4				4	2,230
South Carolina	6	6		9,650	155	9,495		200			200	9,695
South Dakota	4	4		4,320	252	4,068		118			118	4,186
Tennessee	7	7		17,177		17,177		981			981	18,158
Texas	4	4		43,495	5,028	38,467				4	4	38,471
Utah	4	4		3,087		3,087	1			*	1	3,088
Vermont	4	4		2,277		2,277						2,277
Virginia	5	5		15,576	873	14,703	*		11		11	14,714
Washington	5	5		15,612	1,276	14,336	*			9	9	14,345
West Virginia	4	4		7,067	264	6,803	7				7	6,810
Wisconsin	4	4		19,299	1,468	17,831		197			197	18,028
Wyoming	4	4		2,252		2,252	2				2	2,254
District of Columbia	2	2		2,395	13	2,382	11				11	2,393
Total	Weighted average rate 3.85 cents.			723,735	37,104	686,631	233	4,417	38	101	4,789	691,420

[1] Stars indicate amounts less than $500.

[2] Fees for inspection of motor-vehicle fuel. Wherever possible fees for inspection of kerosene and other non-motor-vehicle fuels have been eliminated.

[3] Includes fees for motor-fuel carrier permits, refund or exemption permits, interest on deposits, and miscellaneous unclassified receipts.

[4] A special tax of 3 cents per gallon in Hancock County and 2 cents per gallon in Harrison County is imposed for sea-wall protection. The receipts from these taxes were $130,000 in 1936. These receipts are distributed back to the respective counties.

[5] Ohio imposes a 3-cent tax on motor-vehicle fuel and a 1-cent tax on all liquid fuels. The receipts from the 1-cent tax applicable to non-motor-vehicle fuels (kerosene, fuel oil, etc.) were $689,000. These receipts have been eliminated from the total given, which represents a 4-cent tax on motor-vehicle fuel.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF APRIL 30, 1937

STATE	1/ APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,872,980	$ 51,600	$ 25,800	9.0	$ 1,381,601	$ 690,800	58.5	$ 345,500	$ 172,750	12.1	$ 6,983,630
Arizona	5,394,661	2,220,645	1,717,594	124.3	1,434,196	1,084,980	52.5	404,531	279,269	7.5	2,312,818
Arkansas	6,463,681				2,424,142	2,421,562	148.1	1,310,022	1,309,353	81.5	2,732,766
California	14,366,891	6,264,184	3,560,738	162.2	9,318,237	5,172,708	228.3	1,298,097	729,549	21.8	4,903,896
Colorado	6,911,198	3,972,008	2,109,120	160.1	3,581,611	1,995,118	129.3	443,032	247,012	16.8	2,559,948
Connecticut	2,388,339	771,224	385,224	14.2	745,158	370,176	8.7				1,632,939
Delaware	1,843,750	591,091	286,931	29.3	512,798	256,220	12.3	231,196	115,594	10.4	1,185,005
Florida	5,020,323	1,015,726	501,796	31.5	1,697,855	848,923	57.9	609,970	304,985	7.4	3,364,619
Georgia	9,569,722	1,856,155	886,717	117.7	2,183,984	1,091,977	127.4	1,199,040	599,520	51.0	6,991,508
Idaho	4,635,991	2,645,269	1,534,328	256.6	1,675,377	997,350	102.0	49,381	29,554	7.0	2,074,759
Illinois	15,564,720	8,553,819	4,259,908	136.7	6,816,788	3,353,628	181.4	5,232,697	2,618,487	143.6	5,332,696
Indiana	9,333,269	5,660,722	2,800,122	184.4	4,889,761	2,444,693	115.8	2,370,020	1,185,010	82.2	2,903,445
Iowa	9,757,950	7,219,533	3,513,360	493.8	4,434,057	2,030,570	138.4	1,997,466	871,299	56.9	3,342,721
Kansas	10,005,211	3,798,047	1,896,656	752.2	5,730,694	2,838,900	298.4	3,690,910	1,845,444	176.3	3,424,212
Kentucky	6,961,271	2,151,753	1,038,281	150.6	1,701,912	850,961	41.0	2,046,443	1,023,221	67.6	4,048,808
Louisiana	5,387,420	2,021,801	1,008,034	72.1	1,085,777	542,584	32.3	9,246,580	1,135,750	32.4	2,701,051
Maine	3,299,867	1,911,238	929,566	58.9	1,015,958	507,979	27.0	1,094,759	547,379	31.4	1,314,943
Maryland	3,094,808				1,302,126	651,029	19.9	519,651	259,826	8.1	2,183,954
Massachusetts	5,255,300	333,935	166,968	3.1	4,335,990	2,167,995	20.3	511,780	255,890	2.3	2,664,447
Michigan	11,562,296	9,270,093	4,559,604	332.2	5,972,169	2,985,181	150.9	2,388,450	1,193,550	64.2	2,823,961
Minnesota	10,344,485	8,191,155	3,851,133	532.8	2,621,794	1,308,163	141.7	1,969,147	967,618	90.6	4,217,571
Mississippi	6,635,344	4,640	2,320		1,400,500	700,100	84.5	3,373,670	1,686,040	148.3	4,246,884
Missouri	11,479,090	4,724,554	2,345,944	470.6	7,913,139	3,943,290	269.0	3,658,864	1,774,035	178.8	3,415,820
Montana	7,744,061	4,131,050	2,312,045	409.3	2,869,856	1,607,635	185.4	1,109,912	584,946	74.6	3,239,436
Nebraska	7,809,353	3,253,004	1,623,692	190.8	2,343,573	1,191,149	261.5	2,007,072	1,003,517	251.7	3,990,995
Nevada	4,821,864	1,641,393	1,394,331	272.2	1,673,167	1,445,353	54.4	9,600	8,300	.5	1,973,880
New Hampshire	1,843,750	841,608	406,036	24.8	192,933	96,014	2.3				1,341,700
New Jersey	5,054,295	2,466,551	1,230,502	34.7	2,276,352	1,060,881	24.9	20,880	10,440		2,752,472
New Mexico	6,030,708	3,832,624	2,351,638	309.5	1,673,162	1,017,929	98.3	1,705,313	1,010,055	148.5	1,651,086
New York	18,565,567	8,060,857	3,965,173	167.7	17,536,184	8,141,842	271.1	1,085,400	521,200	22.0	5,937,352
North Carolina	8,877,837	2,494,670	1,243,481	312.9	4,918,404	2,336,652	309.3	2,043,920	932,330	89.2	4,365,375
North Dakota	5,914,683	197,000	105,473	.3	214,800	124,266	2.0	1,135,190	1,135,190	171.8	4,549,754
Ohio	13,771,548	2,473,583	1,231,087	44.3	7,695,049	3,723,521	80.0	2,434,910	1,212,689	27.0	7,604,250
Oklahoma	8,880,547	3,374,231	1,756,840	116.8	2,556,806	1,341,329	90.1	2,215,616	1,164,284	88.9	4,618,094
Oregon	6,182,079	3,002,674	1,794,866	112.8	3,745,458	2,245,576	145.5	1,021,690	557,118	17.6	1,584,518
Pennsylvania	16,129,804	6,673,839	3,308,562	111.2	10,148,433	5,061,176	148.5	2,486,323	1,189,404	27.6	6,570,562
Rhode Island	1,843,750	217,017	106,747	3.4	790,556	395,278	6.6	77,857	36,754	1.1	1,304,972
South Carolina	5,103,525	626,395	262,900	51.7	4,416,239	1,832,523	277.4	777,410	301,702	75.6	2,706,400
South Dakota	6,162,747	1,381,557	775,874	188.6	334,467	194,131	51.7	971,581	536,292	80.7	4,656,451
Tennessee	7,949,380	2,494,552	1,245,146	107.1	1,368,594	684,292	46.0	1,011,028	505,519	40.1	5,514,423
Texas	23,506,431	13,290,909	6,622,202	721.9	10,948,324	5,410,037	736.6	3,624,734	1,783,920	220.7	9,690,272
Utah	4,274,740	2,084,182	1,453,580	138.1	1,131,773	815,447	82.9	125,560	89,533	12.5	1,916,180
Vermont	1,843,750	1,301,001	641,439	62.9	888,145	417,110	26.4	155,340	77,670	5.9	707,531
Virginia	6,887,569	3,158,260	1,575,817	133.3	3,241,793	1,581,803	129.6	1,104,555	552,277	30.0	3,177,671
Washington	5,907,615	4,324,629	2,270,026	164.6	1,744,634	914,371	71.3	1,156,564	557,124	16.5	2,166,093
West Virginia	4,107,201	863,017	431,486	42.3	891,956	445,966	23.9	677,991	335,146	11.6	2,894,604
Wisconsin	9,197,557	3,829,205	1,831,756	160.6	4,250,178	2,070,628	134.0	3,217,170	1,295,675	76.7	3,999,498
Wyoming	4,722,322	3,161,808	1,946,187	392.8	2,150,986	1,320,619	181.7	1,226,440	745,094	161.2	710,421
Puerto Rico	625,000										625,000
Hawaii	1,843,750	29,953	14,542	.8	858,512	424,300	17.2				1,404,908
TOTALS	368,750,000	152,434,761	79,281,572	8,367.7	165,035,958	85,154,715	5,904.2	75,393,262	35,297,314	2,950.2	169,016,399

1/ APPORTIONMENTS FOR FISCAL YEARS 1936 TO 1938 INCLUSIVE.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF APRIL 30, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,314,570	$ 3,299,584	109.2	$ 771,036	$ 771,036	29.1				$ 80,495
Arizona	2,569,841	2,817,718	2,256,486	178.5	354,525	239,795	17.4				73,560
Arkansas	3,352,061	2,721,113	2,702,059	294.7	595,560	594,386	64.5				55,616
California	7,747,928	5,960,206	5,772,207	230.4	2,109,674	1,868,531	29.3	$ 63,950	$ 61,700	4.6	45,490
Colorado	3,395,263	1,819,885	1,763,469	98.1	153,511	153,494	7.0				1,478,300
Connecticut	1,418,709	425,783	406,771	4.3	683,475	656,946	13.2	106,543	88,960	2.8	266,032
Delaware	900,310	497,551	476,418	47.0	355,632	276,827	19.6	19,870	11,076		135,989
Florida	2,597,144	1,905,233	1,871,739	77.7	676,031	676,031	21.4				49,374
Georgia	4,988,967	501,931	491,984	29.4	1,061,480	1,061,480	75.7	857,322	857,322	43.5	2,578,181
Idaho	2,222,747	2,223,133	2,132,180	182.9	56,927	56,927	2.7	7,396	7,396		26,244
Illinois	8,694,009	7,519,354	7,431,432	429.9	1,239,320	1,239,242	41.3				23,335
Indiana	4,941,255	2,953,595	2,802,643	122.8	2,099,308	2,099,308	105.7	37,924	37,924	6.3	1,379
Iowa	4,991,664	3,732,818	3,503,036	424.6	1,492,689	1,432,139	101.2	60,176	44,075	2.4	12,413
Kansas	4,994,975	3,440,405	3,421,727	319.2	1,557,217	1,545,720	59.5	27,528	27,528	9.5	
Kentucky	3,726,271	2,706,939	2,638,223	315.3	689,216	689,216	27.6	314,901	314,892	11.5	83,939
Louisiana	2,890,429	1,406,224	1,251,066	110.2	1,612,242	1,468,068	55.9	171,682	157,053	10.0	14,242
Maine	1,676,799	1,294,221	1,280,861	57.7	317,194	317,194	14.3	62,100	62,100	2.1	16,644
Maryland	1,750,738	478,755	473,816	17.7	465,489	465,489	10.8	475,216	441,384	13.0	370,049
Massachusetts	3,262,885	216,783	216,783	2.5	2,687,114	2,333,754	15.9	812,060	439,338	.4	273,010
Michigan	6,301,414	6,233,848	5,968,828	287.9	336,871	291,871	4.8	5,766	4,650		36,065
Minnesota	5,277,145	5,162,242	4,397,312	815.4	1,147,311	743,326	85.5	137,137	120,248	2.3	16,259
Mississippi	3,457,552	2,173,987	2,169,455	159.6	1,207,208	1,206,169	75.1	8,900	8,900	.6	73,028
Missouri	6,012,652	4,372,992	4,332,974	743.8	1,732,190	1,547,473	32.8	38,755	36,658	1.1	95,547
Montana	3,676,416	3,339,802	3,334,379	185.1	254,250	254,250	10.2	100,210	60,987	.2	26,800
Nebraska	3,870,739	2,591,622	2,517,334	283.8	1,036,387	1,036,383	80.3	67,729	67,729	5.8	249,293
Nevada	2,243,074	1,957,802	1,900,723	89.1	352,584	337,584	13.7	4,767	4,767	5.2	
New Hampshire	945,225	614,516	587,925	26.7	208,060	205,762	8.1	77,464	76,704	2.8	74,834
New Jersey	3,129,805	719,597	716,597	14.5	2,291,862	2,278,707	16.4	105,542	96,573	6.2	37,928
New Mexico	2,871,397	2,253,708	2,249,140	179.2	398,230	398,230	17.0	83,314	83,314	5.2	140,713
New York	11,046,377	9,466,047	9,037,819	134.5	1,657,560	1,657,560	33.4	305,778	305,778	2.4	45,221
North Carolina	4,720,173	2,807,785	2,775,072	208.3	1,872,245	1,834,445	81.0	33,878	27,700	2.3	82,956
North Dakota	2,867,245	1,560,107	1,551,437	231.4	878,804	875,313	90.1	86,471	86,471	16.1	354,024
Ohio	7,670,815	3,305,823	3,241,513	139.8	4,162,047	4,114,612	140.0	193,060	190,540	22.8	124,150
Oklahoma	4,580,670	3,254,994	3,204,702	324.1	1,126,782	1,114,123	69.5	90,798	90,738	2.8	171,107
Oregon	3,038,642	2,046,351	2,025,483	149.3	1,291,232	929,798	12.8	25,003	25,003	5.7	58,358
Pennsylvania	9,347,797	1,831,538	1,741,190	91.1	2,893,643	2,835,918	81.3	3,615,290	3,277,909	93.0	1,492,780
Rhode Island	989,208	1,063,950	973,538	18.8	9,520	9,520		6,520	3,837		2,312
South Carolina	2,702,012	1,451,458	1,377,378	138.7	946,605	921,756	87.4	324,807	277,801	17.6	125,078
South Dakota	2,976,454	1,955,626	1,953,796	383.5	749,357	749,357	80.7	181,040	181,040	29.1	92,261
Tennessee	4,192,460	2,200,158	2,193,276	92.0	1,375,681	1,375,681	43.6	492,461	492,461	13.4	131,042
Texas	11,989,350	12,064,251	11,054,928	1,078.9	896,406	792,468	28.0	76,061	60,475	6.0	81,480
Utah	2,067,154	1,678,191	1,523,294	169.5	385,607	354,243	23.3	76,141	76,141	3.5	113,476
Vermont	924,306	831,474	734,053	20.6	220,948	162,427	1.6	18,212	18,090	.9	9,736
Virginia	3,652,667	2,862,164	2,799,244	927.8	594,789	582,608	103.8	45,533	45,533	4.8	225,282
Washington	3,026,161	3,100,004	2,802,045	162.6	294,845	223,272	1.8				844
West Virginia	2,231,412	645,820	642,324	30.7	1,677,410	1,541,511	64.1	52,460	45,080	2.6	2,497
Wisconsin	4,823,884	4,912,675	4,344,446	328.1	509,195	368,403	15.1	2,999	2,999		108,037
Wyoming	2,219,155	1,656,909	1,654,661	124.2	549,869	546,972	28.4				17,522
District of Columbia	949,496	949,496	949,496	8.8							
Hawaii	926,033	642,788	621,909	8.9	322,161	298,335	8.5				5,789
TOTALS	195,000,000	135,650,042	129,568,755	10,608.8	50,357,299	47,533,660	2,050.4	9,272,764	8,318,874	358.5	9,578,711

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF APRIL 30, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 2,581,622	$ 2,581,622	36	1	6	$ 1,254,847	$ 1,254,847	12			$ 6,220	$ 6,220			6	$ 191,928
Arizona	1,256,099	1,066,867	1,047,303	12			71,181	71,181	1			95,405	95,405	1		3	42,210
Arkansas	3,574,060	1,597,621	1,592,659	35	4	2	1,752,024	1,749,539	20	2		195,718	195,448	1		30	36,415
California	7,486,362	5,126,776	4,961,201	29	8		2,310,757	2,242,680	16			73,800	73,800	6			208,681
Colorado	2,631,567	1,234,005	1,200,507	21	1		939,941	939,940	8								491,121
Connecticut	1,712,684	73,479	73,479		1		779,991	758,630	5		1						880,575
Delaware	418,239	130,000	130,000	1								260,500	260,500	2			27,739
Florida	2,827,883	1,660,162	1,657,852	17	5		826,541	826,198	12	1		79,200	79,200	1			264,634
Georgia	4,895,949	59,085	57,593	1	2		852,182	852,182	17	4		457,310	457,310	10		40	3,528,864
Idaho	1,674,479	906,484	900,019	14	1	4	386,349	386,236	5	1	9	13,339	13,339			8	374,885
Illinois	10,307,184	4,260,678	4,259,716	46	1		5,264,447	5,161,247	27	6		808,000	808,000	3			78,221
Indiana	5,111,096	1,797,167	1,664,428	20	10		3,471,023	3,434,423	22	2	162						12,246
Iowa	5,600,679	2,779,978	2,722,408	68	7	1	2,693,205	2,612,629	37	2	5	229,877	216,070	2		2	49,572
Kansas	5,246,258	2,293,091	2,292,841	35		4	3,010,989	2,936,147	23	1	1						17,271
Kentucky	3,672,387	1,054,606	1,054,606	13	3		1,740,249	1,450,516	9	2		628,328	628,328	4	4		538,936
Louisiana	3,213,467						1,855,395	1,855,395	19	2		988,020	943,290	6	2		414,782
Maine	1,426,861	719,316	718,213	13		2	424,381	423,789	7	1							284,859
Maryland	2,061,751	358,388	358,388	3	3		518,504	518,504	3		19	638,005	615,536	3	2		569,323
Massachusetts	4,210,833	890,234	890,149	7	2		2,144,993	2,144,993	16	2		249,991	249,991	1			925,700
Michigan	6,765,197	3,339,443	3,273,461	34	4		3,454,277	3,454,277	11	4							37,459
Minnesota	5,395,441	2,829,450	2,759,413	59	9	37	2,581,680	2,522,579	26	3	3	58,428	58,428		1	1	55,021
Mississippi	3,241,475	1,035,753	1,035,605	28	3	1	1,550,977	1,550,977	24	1		178,870	178,870	5	1	9	476,023
Missouri	6,142,153	789,025	789,024	14			5,194,702	4,998,457	34	1		367,252	253,300	1		2	101,371
Montana	2,722,327	2,496,141	2,494,225	35	6		178,582	178,582	2			61,025	20,000		1		29,520
Nebraska	3,556,441	1,895,865	1,885,845	65	1	6	1,339,912	1,339,912	14	2	1	274,271	274,271	2		17	56,413
Nevada	887,260	692,129	690,197	7	3		211,658	193,432	1			3,630	3,630			5	1
New Hampshire	822,484	341,748	341,748	3	3		395,145	395,100	5	1	1	8,214	8,214			3	77,422
New Jersey	3,983,826	625,806	625,806	5	2		2,553,369	2,542,324	15	5		125,990	125,990	1			689,706
New Mexico	1,725,286	1,053,490	1,053,475	13	1		673,090	670,400	5								1,410
New York	13,577,189	2,720,770	2,715,945	12	12		9,828,011	9,571,965	28	34		681,000	681,000	3	1		608,279
North Carolina	4,823,958	1,669,440	1,669,440	21	12		2,144,703	2,125,152	27	5	108	203,890	203,890	2			825,476
North Dakota	3,207,473	499,551	499,472	16	1		2,022,837	2,019,557	31	3		312,170	312,170	5			376,274
Ohio	8,439,897	347,806	282,313	2	1	2	5,551,298	5,394,809	37	6		2,669,270	2,345,212	17	1	1	417,562
Oklahoma	5,004,711	1,769,995	1,766,832	33	3		1,857,765	1,857,765	23	4	1	342,033	342,033	3	2	10	1,038,080
Oregon	2,334,204	878,991	867,119	8	5	2	1,491,605	1,410,005	8	1		21,892	21,892	1			35,188
Pennsylvania	11,483,613	1,964,345	1,760,257	26	8		7,501,793	6,993,014	45	8	9	1,341,648	1,212,448	10	4		1,517,894
Rhode Island	699,691	624,125	623,059	4	1		61,626	61,626		2							15,006
South Carolina	3,059,956	830,842	821,976	17	4	3	1,168,759	1,158,495	23	6	3	500,047	497,297	3	4	42	582,188
South Dakota	3,249,086	847,205	847,204	22	2	21	1,578,528	1,578,528	36	3	4	488,555	488,555	8		22	334,799
Tennessee	3,903,979	615,105	615,095	13	1	18	2,466,770	2,466,770	27	2	2	504,240	504,240	6		13	317,874
Texas	10,855,982	5,533,373	5,529,395	86	13	5	4,441,488	4,437,420	43	1	11	484,147	484,143	1		163	405,024
Utah	1,230,763	226,332	224,927	2	1	1	966,695	966,695	15								39,140
Vermont	729,857	499,385	493,786	7	5	14	273,801	229,871	3	2	2	6,200	6,200			4	
Virginia	3,774,287	1,667,608	1,596,742	30	8	8	1,085,338	1,046,340	13	9	2	445,114	439,325	5		10	691,881
Washington	3,095,041	1,931,431	1,924,497	18	9	7	1,194,299	1,158,884	5	3		7,908	7,908			2	3,752
West Virginia	2,677,937						1,701,975	1,700,007	16	3	1	269,672	269,672	6		4	708,258
Wisconsin	5,022,683	2,017,945	1,985,716	25	4		2,977,911	2,886,659	12	3							150,308
Wyoming	1,360,841	540,621	540,513	7			651,776	651,775	5								168,553
Dist. of Columbia	410,804	170,643	170,643	2			254,921	226,161	1								14,000
Hawaii	453,703	170,404	170,389	2			351,976	283,314	3								
TOTALS	196,000,000	69,214,326	68,217,103	987	171	144	98,004,266	95,689,928	797	138	345	14,079,179	13,381,125	119	23	397	18,711,844

U. S. GOVERNMENT PRINTING OFFICE 1937

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

DEPARTMENT BULLETINS

No. 583D..Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

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TECHNICAL BULLETINS

No. 265T...Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP..The Results of Physical Tests of Road-Building Rock. 25 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

No. 191.....Roadside Improvement. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y..Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF APRIL 30, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$15,671,604	$ 8,301,315	$ 3,813,459	770.8	$ 171,795	$ 55,923	$ 115,872	5.8		$ 285,822	1.5	$ 12,895	$ 44,689
Arizona	5,211,960	2,641,935	9,005,933	5,203,675	2,605,032	542.9	24,073		24,073		$ 5,574	810	.1	2,711	12,020
Arkansas	6,748,335	3,428,049	10,969,067	6,627,424	3,371,459	619.9	130,731	80,459	49,922	6.5	33,944	4,200	10.9	6,508	2,469
California	15,607,354	7,932,206	30,631,871	15,607,354	7,735,533	763.9	114,732		114,732						81,941
Colorado	6,874,530	3,486,006	11,269,838	6,874,530	3,465,039	633.2									20,967
Connecticut	2,865,740	1,454,868	4,516,912	2,758,269	1,315,170	74.0	59,618	59,618		.1				47,854	139,697
Delaware	1,819,083	923,395	2,782,833	1,818,604	916,230	130.0						5,400		284	1,765
Florida	5,231,834	2,661,343	9,024,260	5,175,534	2,414,688	307.3	289,764	56,300	233,464	2.9					13,190
Georgia	10,091,185	5,113,491	13,427,069	9,272,425	3,278,773	763.8	1,020,966	327,443	693,523	64.2	131,952	133,980	10.6	359,365	1,007,216
Idaho	4,486,249	2,277,436	7,077,041	4,477,339	2,153,743	501.4	85,967		83,807	.1		1,886		8,910	38,049
Illinois	17,570,770	8,921,401	26,465,811	17,348,739	7,945,886	724.3	1,141,513	129,245	885,007	13.3		7,100		92,786	83,408
Indiana	10,037,843	5,088,963	15,554,741	9,892,557	4,814,845	482.7	227,844	129,083	98,761	2.4		61,727		16,203	113,629
Iowa	10,055,660	5,118,361	15,581,748	10,055,660	4,920,929	1,222.3	209,734		196,600	5.0					832
Kansas	10,089,604	5,117,675	15,426,822	10,089,500	5,009,852	1,132.3	135,923		107,484	13.2				4	339
Kentucky	7,517,359	3,818,311	12,187,003	7,480,730	3,719,296	812.7	67,504		67,504	2.8	11,054	23,834		25,575	7,677
Louisiana	5,828,591	2,963,932	9,135,275	5,765,351	2,694,989	258.9	215,480		215,459	11.5		20,500	6.9	64,940	32,984
Maine	3,369,917	1,711,586	5,245,971	3,346,497	1,671,457	193.5	43,100	15,000	26,540	1.4		800		8,420	12,789
Maryland	3,564,527	1,810,058	5,675,194	3,475,616	1,102,724	152.2	68,343		68,343	1.5		299,044	7.3	88,911	339,947
Massachusetts	6,597,100	3,350,474	10,401,682	6,550,778	3,094,281	115.5						91,944	.1	46,322	164,249
Michigan	12,736,227	6,452,568	20,717,272	12,736,227	6,298,778	768.1	96,788		96,788			40,000	.4		17,002
Minnesota	10,656,569	5,425,551	16,197,923	10,510,789	4,841,459	1,641.9	372,262	58,110	292,626	7.2		216,950	.8	87,669	74,516
Mississippi	6,978,675	3,540,227	12,773,693	6,785,720	3,000,758	724.1	650,119	147,960	488,701	21.4	3,960	14,861	.2	41,035	35,907
Missouri	12,180,306	6,173,740	18,185,313	12,088,271	4,998,938	1,411.2	1,125,699		1,121,765	6.3				92,035	53,036
Montana	7,439,748	3,769,734	11,764,511	7,425,285	3,655,874	1,058.4	56,546		56,546					14,463	57,314
Nebraska	7,828,961	3,964,364	12,970,090	7,812,918	3,746,127	1,035.5	201,659	16,043	185,616	16.4					32,621
Nevada	4,545,917	2,302,356	7,068,575	4,545,917	2,271,616	758.8	21,979		21,979			1,650			7,111
New Hampshire	1,909,839	969,462	2,999,744	1,904,951	949,520	78.3	4,169		4,169					4,888	15,773
New Jersey	6,346,039	3,220,879	9,101,212	6,050,212	2,452,166	85.9	1,313,438	215,511	663,430	11.1		9,212		80,316	96,071
New Mexico	5,792,935	2,941,700	8,898,727	5,749,069	2,910,398	749.9	9,681		9,681			7,047	.2	43,866	14,074
New York	22,330,101	11,327,921	39,972,260	22,118,746	10,637,410	822.6	884,792	199,835	525,499	2.7	5,521	112,200		6,000	52,811
North Carolina	9,522,293	4,840,941	14,951,036	9,205,116	4,516,899	1,345.2	553,051	291,910	261,141	21.5		5,000	.2	25,266	57,902
North Dakota	5,804,448	2,938,967	8,592,708	5,596,661	2,217,011	2,116.1	253,807	137,686	116,121	21.2	43,050	388,620	33.0	27,052	217,215
Ohio	15,484,592	7,865,012	24,759,236	15,378,758	7,524,157	792.1	392,271	90,378	245,354	6.0	8,178	45,000	2.2	7,278	50,501
Oklahoma	9,216,798	4,685,180	14,745,689	9,214,658	4,430,360	806.6	196,489		196,489	.4		31,465		2,141	26,866
Oregon	6,106,896	3,097,814	9,960,993	6,047,235	2,949,337	468.0	64,223		64,223	.8	55,000	12,194	.7	4,061	72,061
Pennsylvania	18,891,004	9,590,788	28,996,588	18,525,500	8,904,961	1,054.0	569,624	192,543	356,300	6.4	146,380	318,502	11.8	26,581	11,025
Rhode Island	1,998,708	1,014,572	3,160,270	1,998,708	1,012,094	89.2	2,478		2,478						
South Carolina	5,459,165	2,770,954	8,025,120	5,220,386	2,472,056	622.0	348,311	148,147	198,079	5.7	40,000	77,808	8.5	50,631	23,011
South Dakota	6,011,479	3,047,643	9,407,360	5,911,486	2,918,908	1,589.5	96,671	5,440	91,231	17.3	51,340	4,390	7.2	43,213	33,114
Tennessee	8,492,619	4,302,991	13,692,251	8,490,706	4,159,606	492.8	115,830		88,814	3.5		46,727	5.9	1,913	7,845
Texas	24,244,024	12,291,253	37,981,565	24,197,559	11,873,156	2,780.7	212,394	2,797	199,611	1.7		185,031	7.5	43,668	33,455
Utah	4,194,708	2,132,691	7,450,900	4,192,008	2,132,691	590.9								2,700	
Vermont	1,867,573	948,007	3,166,359	1,867,452	940,847	141.0								121	7,160
Virginia	7,416,757	3,765,387	11,588,998	7,322,399	3,342,209	621.0	279,711	33,941	240,138	29.1	6,735	10,802	.1	53,681	172,239
Washington	6,115,867	3,106,412	9,397,793	6,112,042	3,026,509	302.7	46,596		46,596			14,823	.3	3,825	18,484
West Virginia	4,474,234	2,280,335	6,479,960	4,334,882	1,806,022	213.3	357,457	54,892	279,519	8.9		66,425	.7	84,460	128,369
Wisconsin	9,724,881	4,941,837	15,457,551	9,724,881	4,888,205	619.7	14,415		14,415			635			38,582
Wyoming	4,501,327	2,287,712	6,931,484	4,457,871	2,235,553	1,040.0	10,551	10,550		.6				32,906	52,159
District of Columbia	1,918,469	973,842	2,887,584	1,918,469	968,979	22.3						4,863			
Hawaii	1,871,062	945,778	2,779,059	1,857,812	362,218	51.8	552,762		535,213	4.1		14,000		13,250	38,347
TOTALS	394,000,000	200,000,000	631,103,599	389,423,791	184,488,707	35,125.2	12,810,860	2,458,814	9,383,613	323.0	542,688	2,565,252	117.1	1,574,707	3,562,428

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 4 JUNE 1937

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 4 June 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

THE VALUE OF PETROGRAPHY IN DETERMINING THE QUALITY OF ROCK

Reported by D. G. RUNNER, Assistant Materials Engineer, Bureau of Public Roads

ALL rocks are constantly being subjected to forces, both chemical and physical, that tend to destroy them. In contrast to these destructive forces, most rocks have many qualities that give them the capacity to withstand this destruction. The capacity of a rock to endure the many destructive agencies is commonly known as durability. At the present time vast quantities of rock are being used daily in the construction of roads and structures, and it is the duty of the engineer to know something of the durability, or "life expectancy" of this material. The science of petrography is of great value in determining the strength and durability of rocks.

Quite often a sample of rock will appear perfectly sound and durable to the eye, while in reality it may contain harmful minerals that are known to be nondurable under freezing and thawing, temperature change, or the decomposing action of other natural weathering agencies. In certain parts of the United States sections of highways, portions of retaining walls, bridge piers, and other types of structures show the effects of the use of nondurable materials (fig. 1). If a petrographic examination of the rocks had been made prior to their use, the inferior materials could have been detected and rejected as unfit for use.

The earliest rocks were solidified from molten magma emanating from the interior of the earth. Upon exposure to weathering and erosion, these igneous rocks in turn have produced sedimentary rocks. Continued burial of igneous and sedimentary rocks, heat, and pressure have altered their composition into the metamorphic type of rock. Elevation of new beds of these rock, together with weathering and erosion, produce new sedimentary rocks, thus these new rocks contain fragments of the original igneous strata. Consequently there exists today a great variety of rocks differing widely in texture, mineralogical character, alteration products, etc. These various types of rock are shown in table 1.

TABLE 1.—*General classification of rocks* [1]

Class	Type	Family
Igneous	Intrusive (plutonic)	Granite.
		Syenite.
		Diorite.
		Gabbro.
		Peridotite.
	Extrusive (volcanic)	Rhyolite.
		Trachyte.
		Andesite.
		Basalt.
		Diabase.
Sedimentary	Calcareous	Limestone.
		Dolomite.
	Siliceous	Shale.
		Sandstone.
		Chert (flint).
Metamorphic	Foliated	Gneiss.
		Schist.
		Amphibolite.
		Slate.
	Nonfoliated	Quartzite.
		Eclogite.
		Marble.

[1] From U. S. Department of Agriculture Bulletin 348, by E. C. E. Lord, 1916.

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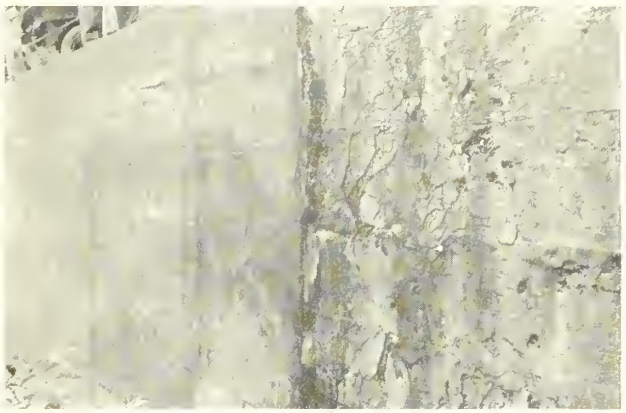

FIGURE 1.—THE LEFT HALF OF THIS CONCRETE RETAINING WALL CONTAINS SATISFACTORY AGGREGATE; THE RIGHT HALF CONTAINS UNSOUND LIMESTONE.

(Photograph by H. S. Mattimore, Pennsylvania Department of Highways)

THIN SECTIONS OF ROCK EXAMINED UNDER THE MICROSCOPE

Petrography may be defined as the descriptive and systematic classification of rocks. This is accomplished with the aid of the petrographic microscope (fig. 2). At this time it might be well to describe briefly how the rocks are prepared for study by this type of microscope. A cursory examination of coarse-grained rocks, such as the granites, enables one to obtain a comprehensive idea of the constituent minerals. However, the minerals contained in fine-grained rocks, such as the basalts, are more difficult to identify. A study of the innermost sections of rock often reveals textural and mineralogical conditions that are interesting and of considerable practical importance.

The first step in making a microscopic examination of a rock is to prepare a small piece so thin that it is transparent to the naked eye. The sample is prepared by first breaking a small chip from the hand sample. This fragment is ground smooth on one side on a revolving lap using an abrasive powder such as emery. (See fig. 3.) The smooth side is cemented to a small glass slide with Canadian balsam and the opposite side is ground smooth. This grinding is continued until the rock slice is about 0.03 millimeter thick, and a thin protective cover glass is then cemented over it. When a specimen is prepared in this manner and examined in the microscope, an accurate idea of the texture and mineral content of the rock can be obtained.

Figure 4 shows a sample data sheet upon which petrographic descriptions of rock specimens are recorded.

All rocks are subjected to weathering by natural agencies, and some materials are more susceptible to this weathering than are others. Each mineral of which a rock is composed has a different rate of decomposition under exposure.

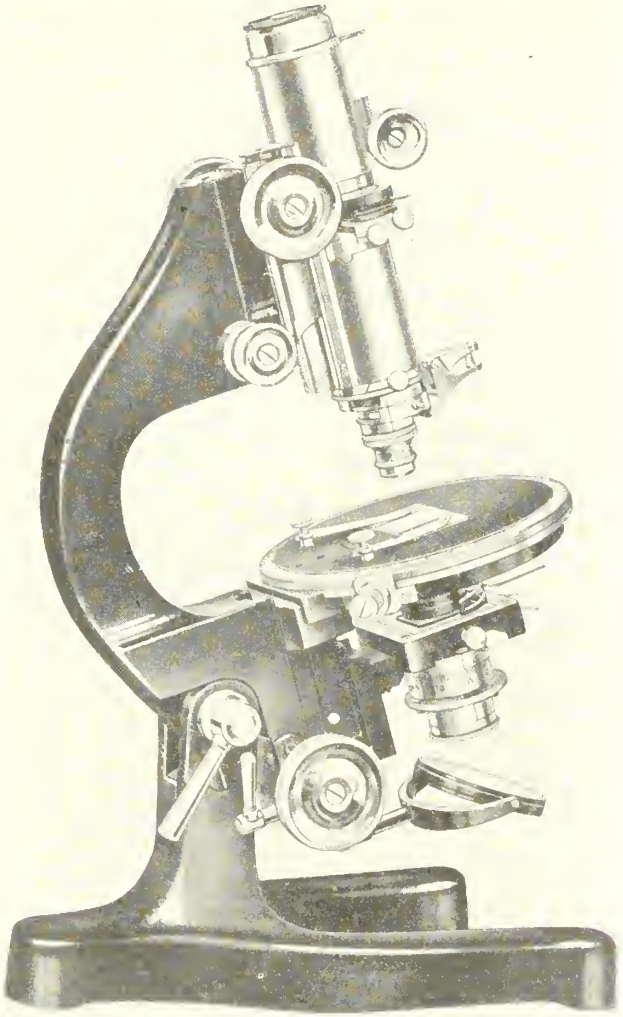

FIGURE 2.—MICROSCOPE USED IN PETROGRAPHIC WORK.

FIGURE 3.—SAW AND GRINDING LAP USED IN PREPARING THIN SECTIONS OF ROCK.

Igneous rocks are composed of several different minerals, while sedimentary rocks consist essentially of one mineral. The minerals of the former are usually interlocking, while those of the latter are united by thin coats of cementing material. Because of inequalities in the rate of expansion of the different particles, stresses are set up in igneous rocks which tend to disrupt the rock. Sedimentary rocks, containing nearly equal-size grains of one mineral, are quite often less injured by temperature changes than are igneous rocks. Other actions that hasten rock decay are: The solvent action of water, carbon dioxide, sulphurous acids, and organic acids; the wedging action of rocks; mechanical abrasion; etc.

Rock strata that have weathered unevenly are illustrated in figure 5. Table 2 gives the alteration products of some common minerals.

TABLE 2.—*Alteration products of some common minerals*

Mineral	Formula	Alteration product
Pyrite	FeS_2	Oxide of iron.
Magnetite	Fe_3O_4	Hematite, limonite.
Ilmenite	$FeTiO_3$	Leucoxene.
Quartz	SiO_2	None.
Enstatite	$MgSiO_3$	Serpentine.
Chrysolite	$(MgFe)_2SiO_4$	Do.
Augite	$RSiO_3$	Chlorite, serpentine.
Hornblende	$RSiO_3$	Chlorite, talc, serpentine.
Biotite (mica)	$(HK)_2(MgFe)_2Al_2(SiO_2)_3$	Chlorite.
Muscovite (mica)	$H_2KAl_3(SiO_4)_3$	None.
Orthoclase (feldspar)	$KAlSi_3O_8$	Kaolin, muscovite.
Plagioclase (feldspar)	$NaAlSi_3O_8$	Kaolin, zeolite.
Sodalite	$3NaAlSiO_4NaCl$	Sericite.
Tremolite		Talc.
Zircon	ZrO_2SiO_2	None.

PYRITE AND FELDSPARS MOST DETRIMENTAL MINERALS FOUND IN GRANITES

Regarding the granites, Washington has stated [1] that:

* * * almost any light-colored, more or less coarse-grained, nonfoliated, igneous rock is called granite by quarrymen, irrespective of its composition, whereas to the petrographer granite denotes a definite species of coarse-grained igneous rock, composed of quartz, alkali (mostly potash), feldspar, and white or black mica or both, in some cases other dark minerals taking the place of the mica. The mineral that is most hurtful to the quality and lasting power of granite is the sulphide of iron, pyrite. On exposure to the action of air and rain this oxidizes, the sulphur forming sulphuric acid, which decomposes the feldspar of the rock and thus disintegrates it, while the iron oxide forms a brown stain. If pyrite is present in granite it is almost always in such small amount and in such small grains that in spite of its usual bright brassy luster, it is seldom visible to the naked eye or by using a hand lens, but its presence is clearly shown under the microscope in the thin section.

Biotite, or black mica, is another mineral common to granites. This mineral alters readily but its effect upon the durability of granite is not so marked as the effect of pyrite and feldspars. Biotite alters in several ways. It may change in color from brown to green while still retaining its micaceous character, the optical constants changing with the chemical change. The most common alteration product is chlorite, but the peculiar cleavage of biotite is retained. Quite often the alteration is accomplished by the deposition of lenticular accumulations of carbonates between the laminae of the mica.

Orthoclase feldspar is another easily altered mineral found in granite. This mineral is composed essentially of potassium-alumina-silicate, with some replacement by sodium. The process of alteration is not likely to be the same in all instances but usually commences along cleavage cracks, and when it has progressed very far the whole mass appears cloudy when viewed through the microscope. The common alteration products are muscovite (sericite), or kaolin, and quartz. Hence the microscopic examination shows at once, by the absence or presence of clouded feldspar, whether the rock is fresh or has begun to weather.

Nephelite, a mineral somewhat akin to the feldspar family and sometimes found in the so-called "granites" quarried for building stone, weathers much more readily than the feldspars and it consequently lessens the durability of the rock. Pyroxene and amphibole

[1] How Petrography Can Aid the Stone Producers, by H. S. Washington, The Explosives Engineer, October 1925.

PETROGRAPHIC DESCRIPTION

I. FIELD NOTES

ORIGINAL NO. 11A LOCALITY Westerly, Rhode Island

OCCURRENCE:

II. HAND SPECIMEN DESCRIPTION

GENERAL APPEARANCE: Fine grained pink rock consisting of feldspar, quartz, and biotite

III. MICROSCOPIC STUDY FOR CLASSIFICATION

TEXTURE: Hypidiomorphic SIZE OF GRAINS:

ORIGINAL STRUCTURE: Cooled from magma

PRIMARY PROCESS REPRESENTED:

SECONDARY STRUCTURE:

SECONDARY PROCESS REPRESENTED:

MINERALOGY (MINERALS ARE GROUPED FOR INTERPRETATION PURPOSES AND ARE ARRANGED IN EACH GROUP IN APPROXIMATE ORDER OF ABUNDANCE)
(IN SOME CASES APPROXIMATE PERCENTAGES ARE GIVEN)

PRIMARY (X) ESSENTIAL MINERALS	%	(Z) SECONDARY ALTERATION PRODUCTS	%	(M) METAMORPHIC RECRYSTALLIZATION MINERALS	%	(T) TERTIARY CHANGES AND ENRICHMENT EFFECTS
Quartz	45	Chlorite				
Plagioclase	10	Sericite				
Orthoclase	15					
Biotite	15					
Muscovite	4					
Microcline	10					
(Y) ACCESSORY MINERALS				(O) INTRODUCED SUBSTANCES OR MINERALIZATION		
Magnetite	1					

SPECIAL FEATURES:

Some of the biotite mica has been bleached into chlorite.

Microcline shows lattice twinning.

ORIGIN OF THE ROCK: Plutonic

CLASSIFICATION: Biotite granite

FIGURE 4.—SAMPLE DATA SHEET UPON WHICH PETROGRAPHIC DESCRIPTIONS OF ROCK SPECIMENS ARE RECORDED.

are more durable constituents than the black micas, although when long exposed they decompose and disintegrate slowly. Those varieties of granite rich in iron weather most readily, and because of large percentages of iron oxide resulting from the decomposition are classed as undesirable.

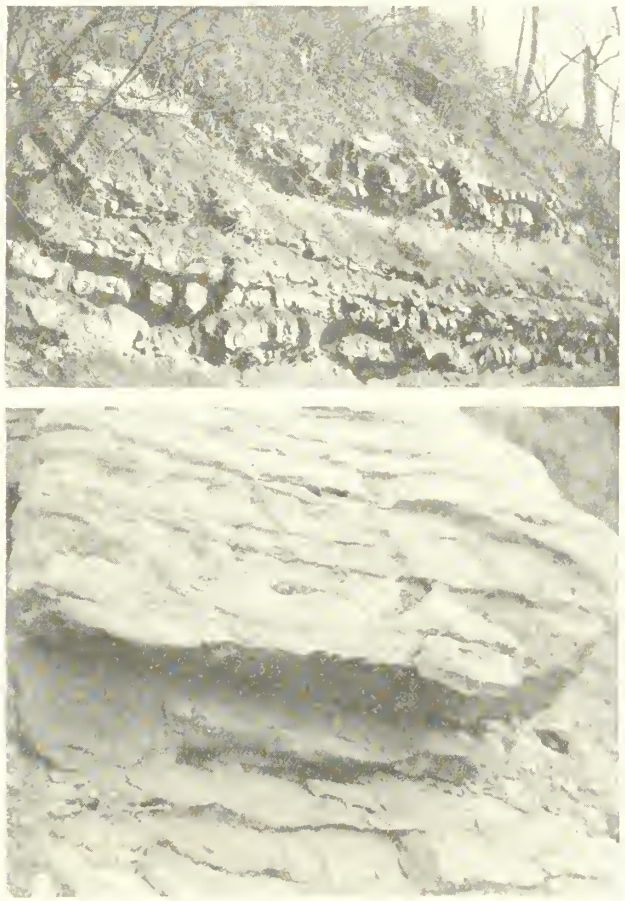

FIGURE 5.—ROCK STRATA THAT HAVE WEATHERED UNEVENLY BECAUSE OF VARIATIONS IN RESISTANCE TO WEATHERING. (UPPER), THE PROJECTING SEAMS CONTAIN THE MORE DURABLE ROCK; (LOWER), DIFFERENTIAL WEATHERING OF LIMESTONE.

Table 3 shows the effect of weathering upon some of the minerals commonly found in granite, and table 4 gives the chemical analyses of three rocks before and after weathering.

CLAY MOST HARMFUL IMPURITY FOUND IN LIMESTONES

Limestones and marbles are examples of rock that to the eye appear to be pure and unadulterated. However, upon examination by the petrographic microscope, they are quite often found to contain minute particles of pyrite or marcasite. Here again the sulphuric acid produced by the oxidation of pyrite (iron disulphide) reacts with the calcium carbonate to cause decomposition of the rock. Marcasite, another iron disulphide, is even more harmful than pyrite because of the rapidity with which it oxidizes and decomposes. The clay contained in some limestones is not visible to the eye but is readily discernible with the microscope. The clay contained by some limestones has often been the cause of unsoundness when used in concrete.

TABLE 3.—*The effect of weathering upon the minerals commonly found in granite*

Mineral	Chemical composition	Changes	Ultimate product
Quartz	SiO_2	Remains undecomposed	Sand grains.
Orthoclase (feldspar).	K_2O	Goes into solution as carbonate, chloride, etc.	Soluble material.
	Al_2O_3, $6SiO_2$	Hydrated and combined to form hydrous aluminum silicate, with liberation of soluble silica.	Clay, soluble material.
Oligoclase (feldspar).	$3Na_2$	Goes into solution as carbonate, chloride, etc.	Soluble material.
	CaO	Forms carbonate, which is soluble in water containing carbon dioxide.	Do.
	$4Al_2O_3$, $20SiO_2$	Decomposes	Clay.
Muscovite (mica).	$2H_2O$, K_2O, $3Al_2O_3$, $6SiO_2$	Remains undecomposed	Mica flakes.
	H_2O		Water.
	K_2O	Goes into solution as carbonate or chloride.	Soluble material.
Biotite (mica)	2(Mg, Fe) O	Goes into solution as carbonate or chloride; iron carbonate oxidizes to hematite or limonite.	Do.
	Al_2O_3, $3SiO_2$	Forms hydrous aluminum silicate and soluble silica.	Clay, soluble material.
Zircon	ZrO_2, SiO_2	Remains unaltered	Zircon grains.
Apatite	$Ca_5(PO_4)_2$, (F, Cl).	Is soluble	Soluble material.

TABLE 4.—*Chemical analyses of rocks before and after alteration by weathering*

Chemical	Micaceous granite		Diabase		Diorite	
	Unweathered	Weathered	Unweathered	Weathered	Unweathered	Weathered
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
SiO_2	69.3	66.8	47.3	44.4	46.8	42.4
Al_2O_3	14.3	15.6	20.2	23.2	17.6	25.5
Fe_2O_3	----------	1.9	3.7	12.7	16.8	19.2
FeO	3.6	1.7	8.9			
MgO	2.4	2.8	3.2	2.8	5.1	.2
CaO	3.2	3.1	7.1	6.0	9.5	.4
Na_2O	2.7	2.6	3.9	3.9	2.6	.6
All others	4.5	5.5	5.7	7.0	1.6	11.7
Total	100.0	100.0	100.0	100.0	100.0	100.0
Loss on ignition	1.2	3.3	2.7	3.7	.9	11.0

Chert is another material that is used quite extensively in certain parts of the United States. It is often found in limestone and dolomite deposits, and is found in many deposits of gravel. Its durability when used in concrete has been questioned by some authorities. Chert is composed essentially of opaline silica together with some impurities such as calcite, pyrite, and organic matter. To the eye, chert appears to contain nothing but silica. However, examination with the microscope sometimes reveals the presence of disseminated pyrite and minute fractures, that may be partly responsible for the reported failures of chert in service.

Kriege has stated: [2]

* * * one of the most common minerals associated with cherts is pyrite. This has been seen in every chert sample examined by the author. In most cases it is microscopically visible, some samples having been found in which the pyrite crystals are several millimeters in length. A very finely divided pyritic mass is sometimes seen coating a chert piece and giving it a brassy green color. More often these sulfide minerals are, as microscopic units, distributed as a specular deposit throughout the mass or in seams and planes. These pyritic bodies soon become oxidized on exposure of the chert and grow into dark iron oxide spots and in time stain the chert mass yellow or brown.

Argillaceous limestones, or those containing fairly large percentages of clay, have been known to cause

[2] The Stability of Chert, by H. F. Kriege, Rock Products, vol. 32, no. 9, Apr 27, 1929.

disintegration when used in concrete. Miller [3] has stated that:

* * * a concrete road and a retaining wall where the stone had been used went to pieces after a few years and the cause was traced to the disintegrating effect of the limestone aggregate. Careful microscopic work showed the presence of considerable argillaceous matter, especially the mineral beidellite, which has adsorptive and absorptive properties. The rock fragments in the concrete road were wet after each rain and with the clay absorbing an undue amount of moisture the rocks were shattered after a limited number of freezings.

Lang [4] has stated that:

* * * use of argillaceous limestones as coarse aggregate in concrete subjected to exposure has probably resulted in more concrete failures than could be attributed to the use of any other unsound type of aggregate.

KIND OF CEMENTING MATERIAL GREATLY AFFECTS DURABILITY OF SANDSTONE

The mineral content has less effect on the quality of sandstones than do the shape of grains and the kind of cementing materials that bind the grains together. Quartz, in more or less rounded grains, is by far the most abundant constituent of sandstones. This is because the chemical processes of weathering do not destroy quartz, except with extreme slowness. Other constituents of sandstones include feldspar, mica, and such accessory minerals as zircon, magnetite, and hematite. Sandstones containing more than about 5 percent of feldspar are usually called "feldspathic sandstone."

The kind of cementing material between the grains of sand greatly affects the durability of the rock. A cementing medium of silica produces a hard rock, while calcium carbonate cement produces one less durable. Argillaceous and ferruginous materials and gypsum may also act as cements. The argillaceous cement does not form a strong bond, and when it is abundant the sandstone tends to break down into sand.

Many sandstones contain layers in which flakes of white mica are abundant. This mica may be mixed with clay material, and may weaken the rock so that it can be easily split into thin slabs. The value of the cementing medium depends chiefly upon its adhesive and cohesive powers. It may be observed that sandstones with silica cement, or limestones with little or no quartz, are relatively strong and durable. Apparently the reason is that the force binding like materials is stronger than that uniting unlike materials.

By means of the microscope the cementing medium can be readily detected and, with the character and shape of the individual constituents determined, the probable durability of the rock can be stated. In discussing the durability of aggregates, Walker [5] has stated that:

Certain shales, soft, loosely bound sandstones, argillaceous sandstones and limestones, ochers, etc., are typcial of unsound aggregate particles causing pits. Their effect on the durability of a concrete structure is approximately in proportion to the amount of pitting which occurs. For ordinary quantities of unsound particles of this nature, the surface pits are usually the only obvious effect. Large proportions of such unsound particles, however, may lead it to progressive disintegration, particularly in the wear surfaces of concrete road slabs.

[3] Limestones of Pennsylvania, by B. L. Miller, Bulletin M20, Pennsylvania Topographic and Geologic Survey, 1934.

[4] Report on Significance of Tests of Concrete and Concrete Aggregates, Proceedings, American Society for Testing Materials, 1935, p. 98.

[5] Report on Significance of Tests of Concrete and Concrete Aggregates, Proceedings, American Society for Testing Materials, 1935, p. 75.

FOLIATED VARIETIES OF METAMORPHIC ROCK APT TO BE UNSATISFACTORY

Marble (crystalline limestone) is usually considered to be the metamorphic equivalent of limestone, chalk, etc. It is quite definitely distinguished from limestone by the crystalline and coarse-grained structure. However, we have dolomitic marbles as well as the calcitic (or limestone) marble, but the distinction is not apparent to the eye, and a chemical or petrographical analysis is usually necessary to bring out the real difference between the two marbles. The probable durability of calcitic and dolomitic marbles may be indicated by knowledge of the accessory minerals present in the original limestone or dolomite. The chief varieties of limestone, based on mineral composition, are named from the accessory minerals listed below:

ACCESSORY MINERAL	VARIETY
Aluminum silicate.	Argillaceous limestone.
Siderite.	Ferruginous limestone.
Detrital quartz.	Arenaceous limestone.
Chalcedony.	Cherty limestone.
Glauconite.	Glauconitic limestone.

A relatively pure limestone, when metamorphosed, may develop into a compact marble, but the impurities shown in the above tabulation produce a broad range of end products. For example, if quartz is the only major impurity, it may react with calcium carbonate to form wollastonite.

Dolomite is essentially a double carbonate of calcium and magnesium. However, this combination is never quite attained to perfection in nature, as there are usually some impurities in the rock. For example, the effect caused by metamorphism on the siliceous or argillaceous impurities may be illustrated by the following equations:

Dolomite + silica = calcite + forsterite + carbon dioxide.

With an increase in the quantity of silica, the following reaction takes place:

Dolomite + silica = diopside + carbon dioxide.

With alumina as an impurity the reaction is as follows:

Dolomite + alumina = spinel + calcite + carbon dioxide.

Common usage of the term marble leads one to believe that there is no difference between marbles; however, petrographically there is a decided difference between the two types of marbles as shown by the impurities in the original limestone or dolomite.

Of the metamorphic rocks, the foliated varieties such as gneiss and schist are most liable to be unsatisfactory. Loughlin has stated [6] that:

* * * the outstanding feature from the standpoint of weathering is the foliated structure. The more finely foliated a rock is, the greater is its tendency to crush into small scaly fragments undesirable for concrete aggregate. Concentration of the micaceous minerals along foliation planes furthermore tends to promote disintegration. The comparatively small amount of mica in gneisses and some schists renders them less subject to disintegration than the highly micaceous schists. Unusually fine-grained schists that appear to consist entirely of mica may exfoliate appreciably after a few months of exposure in humid regions. The beginnings of weathering may extend a considerable distance below the surface of outcrops, and rock that looks satisfactory when newly quarried may soon begin to disintegrate.

[6] Qualifications of Different Kinds of Natural Stone for Concrete Aggregate, by G. F. Loughlin, Proceedings American Concrete Institute, vol. 23, 1927.

DURABILITY OF ROCK PARTLY A FUNCTION OF ITS TEXTURE

The texture of rocks depends upon the shape, size, physical condition, and manner of arrangement of their constituents. The durability of a rock is in part a function of its texture. For example, a coarse-grained sandstone consisting of well-rounded grains is not as durable as an extremely dense, fine-grained sandstone. Likewise a coarse-grained granite is less durable than a dense diabase or basalt.

The uniformity and size of the mineral particles influence the manner in which the rock weathers. For example, in a porphyritic granite the large crystals of feldspar decompose more quickly than the smaller ones, thus pitting the surface of the rock. This differential weathering is usually noted in coarse-grained rocks composed of minerals having various degrees of hardness.

Figure 6 shows photomicrographs of four rocks having different textures.

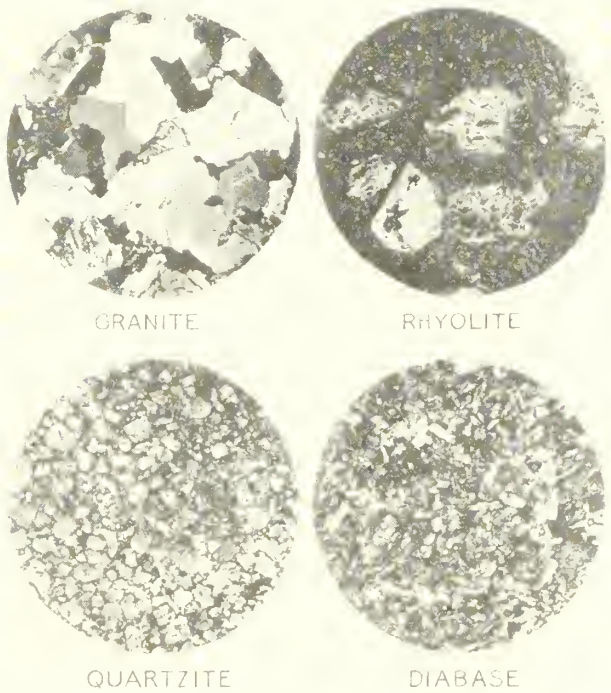

FIGURE 6.—PHOTOMICROGRAPHS OF ROCKS HAVING DIFFERENT TEXTURES.

The grains of sedimentary rocks are usually limited to one size in each particular bed, and may be fine, medium, or coarse. Thus the texture is usually of nearly uniform character, in contrast to the many-sized crystals found in igneous rocks. Because of this, the following discussion will be confined chiefly to igneous rocks.

The texture of a rock depends chiefly upon the mode of formation as shown by the size, shape, and arrangement of the grains of the minerals. The chemical composition of the magma is reflected in the texture of igneous rocks because the original composition determines the minerals that are finally formed. Magmas cooling beneath the earth crystallize slowly. In contrast, the lavas pour out on the earth's surface and cool quickly, sometimes so quickly that they form glass. Between these two extremes of slow and of rapid cooling various rates of cooling occur and cause all gradations of texture.

One common textural term is "granular." This term is applied when most of the constituent minerals are of about the same size, as is often found in the granites. Rocks more basic in composition than the granites have different textures, because of differences in crystallization and the absence of quartz.

"Porphyritic" texture is descriptive of rocks having large crystals, fragments, or flakes contained in a dense ground mass. In such rock structures the crystals have had opportunity to grow unimpeded by other mineral crystals.

"Micrographic" texture refers to the structure produced when quartz simultaneously crystallizes with another mineral, the two interpenetrating and giving a mottled appearance as exemplified by graphic granite.

"Flow" texture is common to the glassy ground mass of extrusive rocks, and has been produced by the cooling of the lava in swirling lines. There are other textural terms such as "ophitic", "cataclastic", and "poikilitic", but these are restricted more or less to special cases and will not be considered here.

SUMMARY

Whenever possible, the engineer should visit the quarry site to note and examine the degree of rock weathering. This examination is particularly advisable in an old quarry where much of the rock to be used has been exposed to the weather. Such features in the rock as laminations, schistose structure, and weathered minerals can be seen with the unaided eye. Weathered feldspar in granites is readily discernible inasmuch as it is usually soft, yellowish to white in color, and is found scattered throughout that portion of the rock directly exposed to weathering. It is not expected that an examination of the quarry will show whether a rock is durable or not, but an examination in connection with a petrographic study will give a fairly definite idea of the lasting quality of the rock.

Loughlin has stated [6] that:

* * * general review of the weathering qualities of natural rocks used as concrete aggregates shows that most of them, if free from weathering, are satisfactory; that certain minerals, particularly the clay group and certain zeolites are very objectionable and that others including micas concentrated in fine grained flaky masses and calcite in finely disseminated grains among other minerals may promote disintegration under certain conditions; that certain textures and structures, notably fragmental texture and shaly structure in sedimentary rocks, flow structure in volcanic rocks, and highly schistose structure in metamorphic rocks, aid in the disintegration of rocks that are mainly composed of durable minerals.

Microscopic study enables an estimate to be made of the probable durability of aggregates. Component minerals can be identified and, having a knowledge of their characteristics, an approximation can be made of the rock's durability. Rock features such as segregations of clay, weathered feldspar, mica, variations in texture, kind of cementing medium, shape of particles, etc., have an important bearing upon the rock's durability.

It is realized that this paper has treated the subject matter in the briefest manner. For more detailed study of the petrographic microscope and the durability of aggregates, the appended list of references may be consulted.

[6] Qualifications of Different Kinds of Natural Stone for Concrete Aggregate, by G. F. Loughlin, Proceedings American Concrete Institute, vol. 23, 1927.

(Continued on p. 77)

VEHICLE SPEEDS ON CONNECTICUT HIGHWAYS

Abstract of report by C. J. TILDEN, Professor of Engineering Mechanics, Yale University [1]

A STUDY to determine the speeds of vehicles on Connecticut highways was made during the period from November 14, 1933, to September 26, 1934, in connection with a highway traffic survey carried on by the United States Bureau of Public Roads and the Connecticut State Highway Department.[2] Observers timed passing vehicles at 78 stations along straight or nearly straight stretches of road. The speeds recorded ranged from that of a heavy truck moving at 9 miles per hour on a wet pavement to the 80-mile-per-hour speed of passenger cars observed on two occasions. The speeds of 91,044 vehicles were measured, and the average was found to be 38.9 miles per hour.

Vehicles were timed by means of a speed detector. This consisted of an L-shaped box, open at each end and painted black on the inside, with a 5- by 7-inch mirror fixed upright across the inner angle of the L. It was mounted on a tripod and set on the roadside in such a position that one end of the box pointed straight across the road, while the other end was directed toward the observer. The mirror was then at an angle to each of these sight lines.

The observer, looking parallel to the road and into the open end of the box, could see in the mirror the reflection of any vehicle on the section of the road directly opposite the box. Each passing car caused a distinct flash or flicker that was readily seen by the observer. The instant the observer saw the flash he pressed the starting button of a stop watch graduated in tenths of a second. At the end of the measured distance (usually 176 feet) the car was reflected in the mirror of another detector, and the observer stopped the watch and recorded the time elapsed. Observations at night were made with the aid of a flashlight set up in such a position that passing cars momentarily cut off the reflection of the light beam in the mirror.

The observations were made from a car parked beside the road. Thus, observers were protected from the weather, while the presence of a car alongside the road aroused less suspicion or curiosity on the part of drivers than would the sight of two men holding stop watches and making notes.

The survey was divided into three main periods: Winter, from November 14 to March 29; spring, from April 4 to June 2; and summer from June 16 to September 10. A summary of the observations made during these periods (table 1) reveals the surprising fact that the average speeds of passenger cars and busses were highest in winter and lowest in summer, despite the fact that 25 percent of the winter observations were made in bad weather. Trucks also made their lowest average speeds in summer, but ran slightly faster in spring than in winter. It seems probable that the slower speed of passenger cars in summer results from the presence of numerous pleasure drivers.

OUT-OF-STATE CARS DRIVEN FASTER THAN CONNECTICUT CARS

The effects of weather and road-surface conditions upon the speed of passenger cars at different seasons were also studied. The results are shown in table 2. The effect of weather on speed ranged from stopping altogether during bad snowstorms to driving at high speeds on clear, crisp winter mornings when the road was free from snow or ice. Practically every kind of weather was encountered during the study. The lowest average passenger-car speed recorded (28.4 miles per hour) was on a clear winter day, when the road was covered with 3 inches of hard-packed snow.

That passenger cars from outside the State are driven at markedly higher rates of speed than Connecticut cars was revealed by the study. The recorded speeds also show that the driver farthest from home drives the fastest. During the summer period, for the daylight hours, cars from Massachusetts and New York averaged, respectively, 40.3 and 41.3 miles per hour, and those from more distant States averaged 41.3 miles

[1] The full report, Motor Vehicle Speeds on Connecticut Highways, has been published by the Committee on Transportation, Yale University. The Bureau of Public Roads does not have copies for distribution.

[2] A digest of the report on the Connecticut traffic survey was published in PUBLIC ROADS, vol. 16, no. 11, January 1936. The Bureau of Public Roads does not have copies of the full report for distribution.

TABLE 1.—*Summary of average speeds of vehicles on Connecticut highways*

Vehicle classification	Winter period (Nov. 14, 1933–Mar. 29, 1934)		Spring period (Apr. 4–June 2, 1934)		Summer period (June 16–Sept. 10, 1934)		Complete survey (Nov. 14, 1933–Sept. 10, 1934)	
	Vehicles observed	Average speed	Vehicles observed	Average speed	Vehicles observed	Average speed	Vehicles observed	Average speed
	Number	*Miles per hour*	*Number*	*Miles per hour*	*Number*	*Miles per hour*	*Number*	*Miles per hour*
Passenger cars (daytime):								
Foreign—								
New York	1,839	45.2	1,259	43.6	6,020	41.3	9,118	42.4
Massachusetts	1,473	45.4	813	44.4	3,435	40.3	5,721	42.2
Other	1,385	45.6	873	44.0	4,928	41.3	7,186	42.5
All foreign	4,697	45.4	2,945	44.0	14,383	41.1	22,025	42.4
Connecticut	14,003	41.4	6,370	39.9	20,273	38.2	40,646	39.6
All passenger cars (daytime)	18,700	42.4	9,315	41.2	34,656	39.4	62,671	40.6
Passenger cars (nighttime)	3,196	37.8	1,821	38.6	5,483	35.2	10,500	36.6
Total all passenger cars	21,896	41.8	11,136	40.8	40,139	38.8	73,171	40.0
All trucks	5,498	34.2	1,746	34.4	4,723	33.0	11,967	33.8
All busses	764	43.5	305	42.5	1,003	39.9	2,072	41.6
Total all vehicles	28,158	40.3	13,187	40.0	45,865	38.2	87,210	39.2

TABLE 2.—*Effect of weather on passenger-car speeds*

Road type	Bad weather conditions				Normal conditions		Decrease in speed	
	Date (1933-34)	Weather and road condition	Vehicles observed	Average speed	Vehicles observed	Average speed		
			Number	*Miles per hour*	*Number*	*Miles per hour*	*Miles per hour*	*Percent*
4-lane concrete	Dec. 11	Snow flurries; snow on road	915	39.2	812	43.7	4.5	10.3
2-lane concrete	Dec. 15	Sleet storm; icy road surface	212	35.4	925	46.3	10.9	23.5
Do	Dec. 27	Clear; road 30 percent snow covered. Some cars using chains	177	35.2	231	36.8	1.6	4.4
Do	Jan. 7	Steady rain	206	37.8	675	43.8	6.0	13.7
Do	Jan. 13	Snow, rain, and slush	81	38.1	103	46.4	8.3	17.9
4-lane concrete	Feb. 5	Clear; snow on road	394	38.8	812	43.7	4.9	11.2
2-lane concrete	Mar. 2	Clear; 3 inches hardpacked snow	182	28.4	298	45.4	17.0	37.5
Macadam	Mar. 8	Light snow	305	36.2	386	40.8	4.6	8.9
2-lane concrete	Mar. 28	Light rain	269	43.1	197	44.7	1.6	3.6
Macadam	June 19	Hard rain	129	34.3				
Do	June 27	Clear; fresh oil on road	147	30.7				
Do	Aug. 7	Dense fog	35	35.3	640	37.1	1.8	4.9
2-lane concrete	Aug. 16	Light rain	324	39.4	205	42.6	3.2	7.5
Macadam	Sept. 6	Dense fog	20	31.4	603	40.8	9.4	23.0

per hour as compared to 38.2 miles per hour for Connecticut cars. Cars from four midwestern States averaged 44.9 miles per hour. The conclusions drawn from these figures are that the driver of the foreign car, because he is farther from home, places a higher value on his time, and that since he is making a longer trip, his car is probably newer or in better mechanical condition, and accordingly can travel faster than the average.

A comparison was made of the daytime and nighttime speeds of passenger cars operated on different types of road surface. The results, shown in table 3, were as anticipated. Daytime speeds were higher than nighttime speeds; and speeds on concrete roads exceeded speeds on macadam roads.

TABLE 3.—*Comparison of daytime and nighttime passenger-car speeds by road type and by season of the year*

Season, and type of road	Daytime observations		Nighttime observations	
	Vehicles observed	Average speed	Vehicles observed	Average speed
	Number	*Miles per hour*	*Number*	*Miles per hour*
Winter (Nov. 14 to Mar. 29):				
2-lane concrete	9,400	42.8	1,108	37.5
4-lane concrete	7,071	42.2	1,858	37.9
Macadam	2,229	41.5	230	37.9
Total or average	18,700	42.4	3,196	37.8
Spring (Apr. 4 to June 2):				
2-lane concrete	4,403	41.6	1,228	39.9
4-lane concrete	4,912	40.8	593	35.9
Total or average	9,315	41.2	1,821	38.6
Summer (June 16 to Sept. 10):				
2-lane concrete	13,591	41.3	1,260	36.2
4-lane concrete	6,348	39.3	2,841	35.4
Macadam	14,717	37.8	1,382	33.6
Total or average	34,656	39.4	5,483	35.2
Complete survey:				
2-lane concrete	27,394	41.9	3,596	37.9
4-lane concrete	18,331	40.8	5,292	36.4
Macadam	16,946	38.2	1,612	34.2
Total or average	62,671	40.6	10,500	36.6

With respect to speed during the day, observations at certain stations from 6 a. m. to 10 p. m. showed that the average speed decreased as the day progressed. The highest speed occurred shortly after daybreak. Speeds then decreased gradually until late afternoon, rose to a minor peak between 5 and 6 p. m., and fell rather sharply after nightfall to the lowest average of the day—about 7 or 8 miles per hour less than the morning peak.

One objective of the survey was to determine the manner in which speed might be affected by traffic volume. Unfortunately, analysis of the data collected indicated that they were inadequate as a basis for any conclusions in this respect. The relationship between speed and volume of traffic, if any exists, depends upon factors such as width and type of pavement, visibility, and general weather conditions.

An answer to the much-debated question of whether women drive faster than men, or vice versa, was sought during two periods of the survey. The conclusion reached was that there is no significant difference. During the period from November 24 to January 31, the men drove 1.1 miles per hour faster than the women; but during the period from July 12 to September 6, the women drove 0.6 mile per hour faster than the men. During the first period, only 9.8 percent of the observed drivers were women; during the second period 17.1 percent of the observed drivers were women.

FAST DRIVERS FOUND TO HAVE WORST ACCIDENT RECORDS

The driving speeds of drivers with and without passengers were studied during these same two periods. It was found that during the winter drivers with passengers drove at a rate only 0.5 mile per hour slower and during the summer they drove 2 miles per hour slower than unaccompanied drivers.

The relation between speed and accidents, a widely discussed and important traffic safety problem, was studied for 6 months during the winter and spring periods. The method used was to select two groups containing nearly the same number of cars: One group included cars observed to be traveling at moderate speeds (35 to 45 miles per hour); the other group included cars whose speeds were 50 miles per hour or more. The observers recorded the license numbers as well as the speeds of these cars. A list of the license numbers was sent to the office of the Connecticut Department of Motor Vehicles, where the name of the owner of each car and his accident record since 1928 were ascertained. Because of duplications, the final figures included 981 cars observed at high speeds and 1,054 observed at moderate speeds.

Results of these observations given in table 4 show that 27.8 percent of the drivers observed traveling at high speeds had been involved in accidents, as compared with 21.3 percent of the drivers observed traveling

at moderate speeds. In other words, 30 percent more of the fast drivers had been in accidents. Moreover, those fast drivers who had been involved in accidents had had more of them and accounted for 45 percent more accidents.

TABLE 4.—*Accident records of owners of cars observed traveling at high and at moderate speeds*

Speed group	Total cars	Owners with accident records	Total accidents	Owners with the following number of accidents—					
				1	2	3	4	5	6
	Number	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*
High	981	273	438	168	71	15	13	5	1
Moderate	1,054	225	324	152	54	13	5	1	0
ACCIDENTS PER THOUSAND CARS									
High	1,000	278	446	171	72	15	13	5	1
Moderate	1,000	213	307	144	51	12	5	1	0

The objections can be raised that since each car was observed only once, a car in the high-speed group may have been going fast only during the interval when it was timed, and vice versa; also, that while the record of the driver was consulted, the owner may not have been the driver at the time the car was observed. The effect of any such errors, however, is to minimize the differences between the records of the two classes of drivers.

Observations were made at a dangerous curve on the Boston Post Road (U S 1) in Madison during 4 days of good weather—2 days in January and 2 in September. Observations were also made during 1 day in February when the road was covered with hard-packed snow. On the days of good weather, the average speed of all the cars observed was 30.4 miles per hour. This included one car that was recorded at 43 miles per hour. On the day when snow covered the road, the average speed was 25.8 miles per hour. For comparison, the speeds of cars were observed during the same 2 days in September on a long straight stretch of road 0.7 mile west of the curve, and were found to be 9.2 miles per hour faster than on the curve.

(Continued from p. 74)

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STATE MOTOR-VEHICLE REGISTRATIONS, 1936

[Compiled from reports of State authorities for calendar year, except as otherwise noted [1]]

State	Registered motor vehicles, private and commercial [2]					Other registered vehicles		Publicly owned vehicles						Dealers' registrations and plates [7]		1935 total registered motor vehicles (revised)	Year's change in motor vehicle registrations	
		Passenger motor vehicles						Federal [5]			State, county, and municipal [6]							
	Total motor vehicles	Total	Automobiles (including taxicabs)	Motor-busses [3]	Motor-trucks, tractor trucks, etc.	Trailers and semi-trailers [4]	Motor-cycles	Motor vehicles	Trailers and semi-trailers	Motor-cycles	Motor vehicles	Trailers and semi-trailers	Motor-cycles	Regular registrations	Extra sets of plates		Increase or decrease	Percentage change
Alabama	297,292	246,557	243,314	3,243	50,735	4,634	720	1,881	40	7	2,969	----------	134	2,889	----------	242,676	54,616	[8] 22.5
Arizona	115,035	94,852	94,473	379	20,183	3,302	345	1,845	51	10	1,727	----------	36	1,813	----------	103,122	11,913	11.6
Arkansas	217,227	169,389	169,079	310	47,838	10,253	417	1,783	15	----------	1,179	15	16	468	----------	207,429	9,798	4.7
California	2,327,984	2,060,533	2,060,533	(9)	267,451	112,088	9,832	5,838	239	83	22,812	1,403	985	4,560	----------	2,151,501	176,483	8.2
Colorado	316,050	284,120	284,120	----------	31,930	1,491	1,108	1,700	12	2	(10)	(10)	(10)	3,151	----------	284,578	31,472	11.1
Connecticut	398,254	338,196	337,216	980	60,058	4,226	1,931	869	3	----------	3,373	46	242	2,752	7,463	366,827	31,427	8.6
Delaware	59,629	49,619	49,619	(9)	10,010	2,227	242	369	2	3	677	9	90	694	----------	56,560	3,069	5.4
Florida	386,907	323,022	321,467	1,555	63,885	13,472	1,000	1,625	32	11	4,971	----------	142	2,502	----------	356,244	30,663	8.6
Georgia	410,583	337,857	337,391	466	72,726	11,903	1,002	2,653	92	43	(10)	(10)	(10)	2,047	----------	394,096	16,487	4.2
Idaho	133,037	107,185	107,060	125	25,852	16,141	472	1,203	65	----------	1,233	27	8	443	----------	118,266	14,771	12.5
Illinois	1,659,750	1,456,652	1,456,652	(9)	203,098	18,168	5,201	3,445	159	19	8,371	240	723	3,963	----------	1,525,817	133,933	8.8
Indiana	905,088	767,279	766,284	995	137,809	52,601	3,443	1,598	144	5	5,095	----------	86	2,385	----------	850,650	54,438	6.4
Iowa	728,414	644,565	644,253	312	83,849	[11] 58,462	2,074	1,252	17	3	[12] 4,880	[12] 215	[12] 56	1,885	----------	699,016	29,398	4.2
Kansas	577,906	490,793	490,793	(9)	87,113	5,070	718	1,443	55	8	(10)	(10)	(10)	1,559	1,165	553,106	24,800	4.5
Kentucky	372,576	320,736	320,138	598	51,840	(13)	866	1,813	19	39	[12] 4,044	----------	----------	826	----------	346,130	26,446	7.6
Louisiana	302,420	228,792	228,361	431	73,628	12,038	776	1,683	32	25	[12] 4,767	[12] 277	[12] 49	348	----------	268,824	33,596	12.5
Maine	191,554	150,606	150,454	152	40,948	[14] 9,124	926	890	6	8	2,228	132	71	748	----------	181,163	10,389	5.7
Maryland	378,462	324,064	323,115	949	54,398	3,224	1,453	2,067	156	46	(10)	(10)	(10)	8,068	----------	345,578	32,884	9.5
Massachusetts	816,711	714,081	709,256	4,825	102,630	10,488	1,319	2,229	57	19	[15] 4,559	----------	----------	2,741	19,668	785,572	31,139	4.0
Michigan	1,373,676	1,234,692	1,234,692	(9)	138,984	119,771	3,233	2,716	46	18	(10)	(10)	(10)	5,400	----------	1,239,431	134,245	10.8
Minnesota	783,627	669,179	668,915	264	114,448	26,567	1,894	2,415	45	8	4,346	----------	----------	(10)	----------	726,993	56,634	7.8
Mississippi	205,890	162,531	159,051	3,480	43,359	1,294	200	1,431	33	----------	(10)	(10)	(10)	2,174	----------	186,289	19,601	10.5
Missouri	809,615	681,644	681,190	454	127,971	26,821	1,631	2,235	16	5	1,423	----------	8	1,872	----------	766,369	43,246	5.6
Montana	167,150	127,839	127,839	(9)	39,311	1,829	416	1,362	7	----------	[12] 1,604	----------	----------	969	----------	149,712	17,438	11.6
Nebraska	413,787	353,192	352,930	262	60,595	26,273	959	992	8	5	1,984	----------	----------	3,109	----------	406,178	7,609	1.9
Nevada	38,509	30,829	30,700	129	7,680	1,060	115	479	20	1	585	26	8	85	453	34,858	3,651	10.5
New Hampshire	122,236	97,361	97,361	(9)	24,875	3,952	969	666	15	----------	389	----------	----------	383	----------	117,154	5,082	4.3
New Jersey	943,412	813,472	808,254	5,218	129,940	5,645	4,739	2,227	77	10	8,678	----------	596	2,599	----------	888,292	55,120	6.2
New Mexico	108,729	85,906	85,427	479	22,823	2,044	288	1,651	59	----------	970	200	12	320	----------	92,457	16,272	17.6
New York	2,453,542	2,134,350	2,134,350	----------	319,192	30,251	10,171	5,780	510	105	[12] 24,458	[12] 847	[12] 987	4,924	----------	2,330,962	122,580	5.3
North Carolina	504,517	434,779	434,194	585	69,738	35,857	1,296	2,247	81	1	731	----------	----------	7,649	----------	463,123	41,394	8.9
North Dakota	167,241	137,591	137,523	68	29,650	517	251	733	3	----------	560	----------	----------	549	----------	164,217	3,024	1.8
Ohio [17]	1,777,048	1,604,775	1,604,775	(9)	172,273	103,308	7,914	2,505	318	9	20,027	----------	----------	3,932	21,540	1,714,627	62,421	3.6
Oklahoma	531,914	441,276	438,803	2,473	90,638	6,960	1,049	2,225	81	9	[12] 5,508	----------	----------	3,386	----------	502,101	29,813	5.9
Oregon	332,729	278,130	277,437	693	54,599	(9)	1,425	2,329	28	1	3,947	----------	----------	653	918	297,112	35,617	12.0
Pennsylvania	1,918,116	1,641,006	1,635,138	5,868	277,110	24,737	12,018	4,254	95	23	(10)	(10)	(10)	29,329	----------	1,745,401	172,715	9.9
Rhode Island	159,140	140,417	139,935	482	18,723	372	672	456	12	----------	1,292	27	99	370	----------	148,597	10,543	7.1
South Carolina	278,829	243,662	243,509	153	35,167	3,320	740	1,258	9	4	3,520	----------	97	664	1,438	235,919	42,910	[18] 18.2
South Dakota	186,480	158,264	158,192	72	28,216	19,349	409	1,048	13	2	1,021	109	9	592	----------	179,271	7,209	4.0
Tennessee	380,792	329,405	328,955	450	51,387	[19] 7	1,348	1,660	31	----------	4,671	----------	----------	552	----------	351,898	28,894	8.2
Texas	1,478,124	1,192,147	1,191,313	834	285,977	41,660	3,544	4,459	192	79	13,897	1,083	347	3,651	----------	1,382,104	96,020	6.9
Utah	116,816	97,419	96,768	651	19,397	1,066	414	1,573	44	11	1,212	86	55	1,186	----------	106,006	10,810	10.2
Vermont	84,155	75,310	75,195	115	[20] 8,845	1,464	542	937	13	1	(10)	(10)	(10)	573	----------	81,513	2,642	3.2
Virginia	417,463	352,281	352,281	----------	65,182	6,359	1,761	3,288	203	63	[12] 5,058	[12] 116	[12] 150	5,021	----------	385,555	31,908	8.3
Washington	499,760	420,222	419,493	729	79,538	12,122	1,775	3,283	59	40	6,335	257	171	1,217	2,778	453,660	46,100	10.2
West Virginia	280,015	236,532	235,913	619	43,483	2,721	1,144	1,071	18	----------	4,425	147	58	11,545	----------	248,379	31,636	12.7
Wisconsin	835,178	690,525	690,041	484	144,653	4,736	2,852	2,345	18	6	7,480	139	295	2,377	----------	754,037	81,141	10.8
Wyoming	76,603	61,129	61,129	----------	15,474	8,542	220	941	31	9	624	30	----------	313	----------	69,998	6,605	9.4
District of Columbia	181,319	162,922	161,725	1,197	18,397	1,843	707	1,310	23	44	2,600	118	202	3,609	----------	171,464	9,855	5.7
At large								2,976	29	10								
Total	28,221,291	24,197,685	24,156,606	41,079	4,023,606	869,359	98,541	99,038	3,333	795	200,230	5,549	5,732	142,845	55,423	26,230,834	1,990,457	7.6

[1] Registration periods ending not earlier than Nov. 30 and not later than Jan. 31 are considered as calendar-year periods. In the case of States in which the registration period is definitely removed from the calendar year registration figures were obtained for the calendar-year period. The figures for Ohio, however, represent registrations for the 9-month period ended Dec. 31, 1936.

[2] Wherever possible publicly owned vehicles and vehicles not for highway use have been eliminated from these columns.

[3] A complete segregation of motor busses from other vehicles is not available. The figures given represent common-carrier busses in most cases, although in some States contract busses and contract school busses are included. In a number of cases city busses are not included, rural and interurban carriers only being given. Where no busses are tabulated the busses are included with automobiles, except as otherwise noted.

[4] Figures for trailers and semitrailers are as reported. Apparent inconsistencies are due to the fact that some States require the registration of tourist trailers, light work trailers, and similar vehicles, whereas other States register only freight-carrying trailers and semitrailers.

[5] Preliminary figures based on incomplete data for 1936 and complete data for 1935, obtained through agency of Procurement Division, Department of the Treasury.

[6] Data on State, county, and municipal vehicles are incomplete in many cases. Some States give State-owned vehicles only; others exclude certain classes, such as fire apparatus and police vehicles, from registration.

[7] Figures include new-car, used-car, and motorcycle dealers' registrations, and in some cases wreckers' and repairers' registrations. Data on dealers' extra plates incomplete, although extra plates are apparently included with dealers' registrations in some cases.

[8] Large increase due to fact that 1935 figures represent registrations during fiscal year ended Sept. 30, 1935, whereas 1936 figures represent registrations during calendar year 1936.

[9] Included with motor trucks.

[10] Included with private and commercial registrations.

[11] Includes 53,551 light trailers registered without charge.

[12] Includes unknown number of Federal vehicles.

[13] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as trucks. Light trailers permitted but not registered.

[14] Includes light trailers and commercial semitrailers. Commercial full trailers included with motor trucks.

[15] Of these vehicles approximately 1,500 are included with private and commercial registrations.

[16] Not reported.

[17] For 9-month period ended Dec. 31, 1936. Data for full calendar year not available.

[18] Large increase due to fact that 1935 figures represent registrations during fiscal year ended Oct. 31, 1935, whereas 1936 figures represent registrations during calendar year 1936.

[19] Trailers for passenger vehicles only. Freight trailers registered with trucks.

[20] Light delivery trucks included with passenger cars.

STATE MOTOR-VEHICLE RECEIPTS, 1936

[Compiled from reports of State authorities for calendar year, except as otherwise noted [1]]

State	Total receipts, registration and other fees	Motor-vehicle registration fees					Registration fees, other vehicles		Total registration fees, all vehicles	Miscellaneous receipts							
		Total [2]	Passenger motor vehicles			Motor trucks, tractor trucks, etc.	Trailers and semi-trailers	Motorcycles		Total	Dealers' licenses and plates	Operators' and chauffeurs' permits	Certificates of title	Fines and penalties	Transfer or registration fees	Miscellaneous receipts, less unclassified refunds [4]	Estimated service charges, local collectors [5]
			Total	Automobiles (including taxicabs)	Motor busses [3]												
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	4,101	3,777							3,777	324	[6]	227		88		9	
Arizona	1,028	754	365	337	28	389	48	1	803	225	3	35	88	2	36	61	
Arkansas	2,823	2,361	1,287	1,232	55	1,074	262	1	2,624	205	18	123		7	8	49	
California	21,087	18,758	14,713	14,713	[7]	4,045	784	45	19,587	1,500	47	176		71	1,128	78	
Colorado	2,589	1,893	1,436	1,436		457	26	2	1,921	668	24	344	195	4	91	10	
Connecticut	5,957	3,904	2,580	2,470	110	1,324	15	6	3,925	2,032	102	1,546		195	123	66	
Delaware	1,102	804	544	544	[7]	260	25	1	830	272	6	123	38	67	16	22	
Florida	5,546	5,017	3,539	3,396	143	1,478	180	5	5,202	344	21	8	203		1	13	98
Georgia	1,302	1,230	1,013	1,013		217	35	3	1,268	34	6	11			8	9	
Idaho	2,175	2,066	1,478	1,454	24	588	29	2	2,097	78	26	41				11	
Illinois	19,410	17,612	12,339	12,339	[7]	5,273	143	14	17,769	1,641	87	314	501		639	100	
Indiana	9,044	7,133	5,330	5,243	87	1,803	290	6	7,429	1,615	46	566	352		484	−35	202
Iowa	10,793	10,297	7,713	7,713		2,584	74	7	10,378	415	47	78		1	203	86	
Kansas	3,815	3,584	2,769	2,769	[7]	815	66	4	3,654	161	22	40			73	26	
Kentucky	4,591	3,262	2,146	2,102	44	1,116	[8]	2	3,264	1,327	24	391		1	107	482	322
Louisiana	4,112	3,605	2,531	2,480	51	1,074	418	3	4,026	86	13	77		9		−13	
Maine	3,582	2,703	1,949	1,935	14	754	[9] 54	5	2,762	820	58	523		21	156	62	
Maryland	4,744	3,141	2,566	2,413	153	575	93	7	3,241	1,503	42	303	716	275	127	40	
Massachusetts	6,795	4,188	2,699	2,578	121	1,489	19	2	4,209	2,586	67	2,442		11		66	
Michigan	19,737	16,768	11,697	11,697	[7]	5,071	854	12	17,634	2,103	71	673	854		369	136	
Minnesota	8,189	7,691	5,861	5,740	121	1,830	225	6	7,922	267	47	126	1	68		25	
Mississippi	1,869	1,830							1,830	39	2			33		4	
Missouri	8,988	7,832	6,560	6,560	[7]	1,272	110	9	7,951	1,037	77	206	437	2	293	22	
Montana [10]	1,730	1,480	1,087	1,087	[7]	393	5	1	1,486	244	19	112	34	30	43	6	
Nebraska	2,158	1,951	1,219	1,198	21	732	34	2	1,987	171	15	26			124	6	
Nevada	279	252	154	153	1	98	9	1	262	17	3		1	[*]	11	2	
New Hampshire	2,635	2,149						5	2,154	481	23	357		88		13	
New Jersey	17,850	12,641	8,817	8,522	295	3,824	209	9	12,859	4,991	65	3,582	547	146	557	94	
New Mexico	1,318	1,217	819	789	30	398	17	1	1,235	83	7			1	23	52	
New York	46,291	40,443	29,190	29,190		11,253	466	46	40,955	5,336	196	3,188		453	1,217	282	
North Carolina	7,589	7,350	4,820	4,780	40	2,530	[7]	5	7,355	234	[6]	136	163			−65	
North Dakota	1,456	1,338	1,017	1,008	9	321	3	1	1,342	114	16	41	56			1	
Ohio [11]	23,256	21,029	13,439	13,439	[7]	7,590	1,020		22,049	1,207	174	651				68	314
Oklahoma	4,743	3,700	2,404	2,341	63	1,296	130	4	3,834	909	43		295			271	300
Oregon	2,832	2,475	1,422	1,388	34	1,053	[7]	5	2,480	352	21	85	190	27		29	
Pennsylvania	35,331	25,626	17,518	16,944	574	8,108	313	29	25,968	9,363	347	5,204	1,958	471	1,179	204	
Rhode Island	2,596	2,045	1,583	1,537	46	462	2	2	2,049	547	16	397			90	44	
South Carolina	1,876	1,523	1,103	1,103		420	50	1	1,574	302	17	178		40	19	48	
South Dakota	1,540	1,464	1,264	1,262	2	200	27	[*]	1,491	49	15		25			9	
Tennessee	3,706	3,526							3,526	180	[6]			35		12	133
Texas	17,725	16,711	10,982	10,856	126	5,729	390	16	17,117	608	44	186			209	169	
Utah	976	808	484	484	[7]	324	22	1	831	145	12	17	66	5	25	20	
Vermont	2,245	1,899	1,348	1,332	16	[12] 551	8	4	1,911	334	25	268			50	−9	
Virginia	5,737	5,075	3,724	3,724		1,351	48	5	5,128	609	55	82	246		171	55	
Washington	2,980	2,403	1,405	1,341	64	998	162	6	2,571	409	12	111	141			15	130
West Virginia	5,832	4,438	3,396	3,376	20	1,042	53	6	4,497	1,335	48	196	1,001		66	24	
Wisconsin	12,213	11,588	8,552	8,373	179	3,036	317	14	11,919	294	28	35			219	12	
Wyoming	541	501	336	336		165	25	[*]	526	15	6				8	1	
District of Columbia	963	186	163	162	1	23	2	1	189	774	5	287	103	329	37	13	
Partial totals [13]			207,361	204,889	2,472	85,385	7,062	308									
Full totals	359,783	304,028							311,398	48,385	2,068	23,512	8,211	2,480	7,910	2,705	1,499

* Receipts less than $500.

[1] Registration periods ending not earlier than Nov. 30 and not later than Jan. 31 are considered as calendar-year periods. In the case of States in which the registration period is definitely removed from the calendar year, data on receipts were obtained for the calendar-year period. The figures for Ohio, however, represent receipts for the 9-month period ended Dec. 31, 1936.

[2] No segregation of registration fees by type of vehicle was available in Alabama, Mississippi, New Hampshire, and Tennessee. For these States the total motor-vehicle registration fees include those of trailers and motorcycles, except in the case of New Hampshire, for which motorcycle fees were reported separately. Dealers' license fees in Alabama and Tennessee are also included.

[3] The figures for registration fees of motor busses are incomplete. (See footnote 3 of preceding table.) Where no fees are tabulated, the fees of busses are included with those of automobiles, except as otherwise noted.

[4] Deduction of refunds results in a negative item in a number of cases.

[5] In a large number of States service charges are collected or deducted by the county or local officers who issue registrations. In the majority of cases these charges are included in the registration and other fees as listed. The amounts shown in this column are estimates of service charges collected and retained by local officials and not reported elsewhere in the table.

[6] Included with motor-vehicle registration fees.

[7] Included with fees of motor trucks.

[8] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as trucks. Light trailers permitted but not registered.

[9] Fees of light trailers and commercial semitrailers only. Fees of commercial full trailers included with those of motor trucks.

[10] Registration fees are collected by counties, and State does not maintain complete record. Figures given are estimates supplied by State.

[11] For 9-month period ended Dec. 31, 1936. Data for full calendar year not available.

[12] Fees of light delivery trucks included with those of automobiles.

[13] Totals of columns for which fully classified totals were not available for all States.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MAY 31, 1937

STATE	1/ APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,872,980	$ 51,600	$ 25,800	9.0	$ 1,354,601	$ 677,300	58.5	$ 667,250	$ 333,625	37.2	$ 6,836,255
Arizona	5,394,661	2,578,176	1,972,485	145.1	1,049,303	811,285	31.7	880,898	573,145	23.3	2,037,746
Arkansas	6,463,681				3,084,246	3,081,271	184.0	870,040	869,086	68.8	2,513,324
California	14,366,891	7,459,280	4,225,587	196.6	8,176,442	4,573,646	197.4	2,595,986	1,318,052	57.6	4,249,606
Colorado	6,911,198	4,506,541	2,407,190	176.3	3,395,599	1,890,122	128.3	439,333	240,544	4.7	2,373,342
Connecticut	2,388,339	771,388	384,238	14.2	745,158	370,176	8.7				1,633,925
Delaware	1,843,750	688,441	333,428	33.0	469,148	234,444	12.3	316,098	155,861	20.5	1,120,018
Florida	5,020,323	1,022,926	505,396	31.5	1,690,655	845,323	57.9	609,970	304,985	7.4	3,364,619
Georgia	9,569,722	1,856,155	886,717	117.7	2,519,984	1,259,977	136.5	1,921,920	960,960	103.6	6,462,068
Idaho	4,635,991	2,825,120	1,632,661	263.4	1,594,295	953,976	104.4	483,186	287,966	39.4	1,761,388
Illinois	15,564,720	8,864,008	4,404,074	139.1	7,989,182	3,944,825	212.5	6,869,378	3,431,827	196.2	3,783,993
Indiana	9,333,269	5,968,995	2,942,550	197.4	5,855,691	2,927,736	150.3	1,896,120	948,060	49.1	2,514,924
Iowa	9,757,950	7,200,433	3,536,382	495.8	4,515,529	2,059,231	140.8	2,175,612	946,059	62.4	3,216,278
Kansas	10,005,211	4,732,267	2,364,033	833.9	6,478,352	3,212,729	263.3	2,124,113	1,062,046	143.6	3,366,403
Kentucky	6,961,271	2,249,866	1,085,244	152.5	2,418,979	1,137,899	63.2	2,277,934	1,138,967	81.6	3,599,161
Louisiana	5,387,420	2,014,682	1,004,304	72.1	1,085,777	542,584	32.3	9,246,580	1,135,750	32.4	2,704,781
Maine	3,299,867	1,911,539	933,106	58.9	1,636,665	818,333	49.5	637,830	318,915	15.6	1,229,513
Maryland	3,094,808				1,305,676	652,804	19.9	516,051	258,026	8.0	2,183,979
Massachusetts	5,255,300	333,935	166,968	3.1	4,329,066	2,164,533	20.3	511,780	255,890	2.3	2,667,909
Michigan	11,562,296	9,288,948	4,548,784	332.2	6,120,169	3,059,181	154.4	3,250,100	1,614,449	76.2	2,339,882
Minnesota	10,344,485	8,207,986	3,861,650	537.3	3,335,995	1,648,314	187.6	1,719,765	859,877	69.9	3,974,644
Mississippi	6,635,344	4,640	2,320		1,737,210	868,500	100.1	3,853,890	1,925,740	173.7	3,838,784
Missouri	11,479,090	4,914,088	2,440,070	477.5	8,044,731	4,009,086	293.0	3,732,924	1,648,903	185.8	3,381,031
Montana	7,744,061	4,131,050	2,247,844	409.3	3,166,965	1,774,575	215.7	1,091,040	493,457	63.2	3,228,185
Nebraska	7,809,353	3,225,232	1,598,845	190.8	3,233,957	1,636,341	370.6	1,700,042	850,021	195.0	3,724,146
Nevada	4,821,864	1,642,101	1,394,112	272.2	1,766,518	1,526,190	54.9	258,341	216,907	37.6	1,684,655
New Hampshire	1,843,750	850,588	404,321	24.8	194,187	96,641	2.3				1,342,788
New Jersey	5,054,295	2,667,645	1,327,644	39.3	2,075,259	960,334	20.3	20,880	10,440		2,755,877
New Mexico	6,030,708	3,997,848	2,452,029	334.6	1,969,311	1,157,131	115.7	1,726,512	1,170,521	115.5	1,251,027
New York	18,565,567	8,220,786	3,995,419	169.9	18,308,914	8,530,207	290.5	2,795,600	1,363,850	43.3	4,676,091
North Carolina	8,877,837	2,944,435	1,467,210	347.5	4,942,372	2,348,636	335.4	2,189,840	1,022,670	72.6	4,039,322
North Dakota	5,914,683	197,000	105,473	.3	366,260	275,716	25.3	1,493,570	1,493,570	238.7	4,039,924
Ohio	13,771,548	2,965,171	1,466,760	47.4	7,627,261	3,689,627	79.7	3,080,610	1,535,539	40.6	7,079,621
Oklahoma	8,880,547	3,958,051	2,063,516	144.0	2,394,352	1,255,960	91.4	2,889,546	1,498,783	101.7	4,062,288
Oregon	6,182,079	3,083,627	1,853,241	117.5	4,588,881	2,705,023	157.6	500,528	285,325	29.9	1,338,489
Pennsylvania	16,129,804	6,533,546	3,226,952	110.4	10,942,129	5,458,961	157.5	1,845,158	919,546	20.5	6,524,345
Rhode Island	1,843,750	215,607	104,733	3.3	794,476	397,238	6.6	346,360	170,310	5.8	1,171,469
South Carolina	5,103,525	561,512	246,900	53.8	4,957,391	2,050,523	317.3	659,435	298,700	36.8	2,507,402
South Dakota	6,162,747	1,315,271	739,076	176.7	1,173,903	657,691	108.5	795,324	442,723	105.6	4,323,257
Tennessee	7,949,380	2,664,334	1,311,738	115.0	1,844,922	922,461	59.9	634,540	317,270	20.4	5,397,911
Texas	23,506,431	13,670,181	6,810,279	778.2	11,912,022	5,898,003	803.6	3,737,350	1,765,643	179.5	9,032,507
Utah	4,274,740	2,191,492	1,530,051	144.8	1,047,653	755,719	77.9	512,870	344,790	56.6	1,644,180
Vermont	1,843,750	1,301,605	641,741	62.9	1,043,485	494,780	32.2	60,200	30,100	1.2	677,129
Virginia	6,887,569	3,503,837	1,745,178	146.3	3,169,962	1,545,891	130.7	879,960	439,980	26.1	3,156,520
Washington	5,907,615	4,373,407	2,258,713	164.6	1,973,371	1,034,271	76.2	922,679	481,098	11.8	2,133,533
West Virginia	4,107,201	863,017	431,486	42.3	900,256	450,116	23.9	1,124,578	558,439	26.8	2,667,160
Wisconsin	9,197,557	3,976,182	1,913,908	167.7	5,380,511	2,444,448	165.2	4,192,342	1,730,182	96.8	3,109,019
Wyoming	4,722,322	3,041,068	1,870,594	399.1	2,992,166	1,840,849	263.1	790,370	463,017	71.4	547,861
Puerto Rico	625,000										625,000
Hawaii	1,843,750	29,953	14,542	.8	855,062	420,545	17.2	124,030	58,735	.8	1,349,928
TOTALS	368,750,000	159,575,560	82,885,292	8,750.1	178,553,999	92,071,152	6,606.1	81,968,463	38,550,349	3,057.5	155,243,207

1/ APPORTIONMENTS FOR FISCAL YEARS 1936 TO 1938 INCLUSIVE.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MAY 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,321,378	$ 3,306,391	109.2	$ 771,036	$ 771,036	29.1				$ 73,687
Arizona	2,569,841	2,797,425	2,256,485	178.5	357,455	242,725	17.3				70,631
Arkansas	3,352,061	2,791,353	2,768,637	303.8	532,396	531,222	55.3	$ 16,282	$ 15,714	1.2	36,489
California	7,747,928	6,300,614	6,113,621	233.7	1,831,563	1,624,227	30.6	7,200	7,200		2,880
Colorado	3,395,263	1,889,495	1,831,891	99.1	89,597	89,596	6.0				1,473,776
Connecticut	1,418,709	580,639	534,508	4.4	537,218	517,407	13.2	84,583	82,500	2.8	284,294
Delaware	900,310	605,917	580,185	48.9	247,538	247,538	17.7	37,945	29,151	.2	43,436
Florida	2,597,144	1,970,774	1,937,280	77.9	608,719	608,719	21.2				51,145
Georgia	4,988,967	671,162	661,215	39.6	1,033,689	1,033,689	75.9	769,552	769,552	36.4	2,524,511
Idaho	2,222,747	2,242,791	2,151,838	185.3	44,665	44,665	.3				26,244
Illinois	8,694,009	7,819,498	7,706,054	440.1	950,754	950,676	31.1				37,279
Indiana	4,941,255	2,985,205	2,795,259	122.8	2,137,232	2,137,232	112.0				8,764
Iowa	4,991,664	3,715,647	3,485,929	424.6	1,513,751	1,433,134	103.0	44,378	43,070	.4	29,531
Kansas	4,994,975	3,885,339	3,855,480	325.4	1,124,880	1,062,708	50.2	55,360	55,360	14.2	21,427
Kentucky	3,726,271	2,850,194	2,758,762	317.9	613,277	613,277	28.5	268,879	268,879	7.8	85,353
Louisiana	2,890,429	1,767,022	1,587,428	129.7	1,330,755	1,196,389	38.0	106,613	106,613	3.6	
Maine	1,676,799	1,303,328	1,289,370	59.3	323,032	323,032	13.3	52,100	52,100	1.8	12,296
Maryland	1,750,738	478,755	473,816	17.7	730,571	730,571	17.0	210,134	176,302	6.9	370,049
Massachusetts	3,262,885	216,783	216,783	2.5	2,687,114	2,333,754	15.9	816,540	443,818	.4	268,530
Michigan	6,301,414	6,325,701	5,965,311	287.9	291,871	291,871	4.8	11,466	10,350		33,882
Minnesota	5,277,145	5,186,071	4,411,073	823.2	1,130,865	734,783	77.7	131,648	114,798	2.3	16,492
Mississippi	3,457,552	2,242,832	2,238,300	164.0	1,105,017	1,103,977	70.7	8,900	8,900	.6	106,375
Missouri	6,012,652	4,441,704	4,399,532	747.1	1,666,075	1,481,358	29.6	37,060	34,964	1.1	96,798
Montana	3,676,416	3,375,707	3,368,616	195.1	249,598	249,598	.3	215,634	53,538	9.8	4,665
Nebraska	3,870,739	3,086,730	3,007,972	323.7	573,739	573,736	43.8	20,461	20,461	2.0	268,571
Nevada	2,243,074	2,030,445	1,969,541	89.1	279,244	264,244	13.7	9,289	9,289	5.2	
New Hampshire	945,225	615,425	587,476	26.7	285,524	282,466	10.9	3,812	3,812		71,471
New Jersey	3,129,805	832,403	829,403	15.3	2,222,746	2,209,591	18.8	56,533	56,533	2.0	34,278
New Mexico	2,871,397	2,252,349	2,247,612	179.2	398,230	398,230	17.0	139,585	139,585	7.5	85,970
New York	11,046,377	9,703,228	9,224,042	136.1	1,599,860	1,599,860	34.2	37,477	37,477		184,997
North Carolina	4,720,173	2,946,831	2,914,118	209.0	1,799,730	1,758,852	81.2	1,600	1,600	.1	45,603
North Dakota	2,867,245	1,675,425	1,661,201	248.7	735,375	735,220	87.6	281,120	281,120	54.1	189,704
Ohio	7,670,815	3,324,629	3,260,320	145.0	4,177,467	4,130,032	143.6	189,640	187,120	10.9	93,344
Oklahoma	4,580,670	3,449,175	3,382,040	338.3	960,857	960,272	57.2	248,332	203,732	12.7	34,625
Oregon	3,038,642	2,046,351	2,025,483	149.3	1,294,918	933,484	15.3				79,675
Pennsylvania	9,347,797	2,040,064	1,939,709	97.1	3,669,408	3,616,608	102.0	3,572,023	3,024,650	79.7	766,830
Rhode Island	989,208	1,073,470	983,058	18.8				6,520	3,837		2,312
South Carolina	2,702,012	1,507,295	1,418,056	142.5	1,035,143	1,015,319	90.1	190,564	143,558	11.2	125,079
South Dakota	2,976,454	2,095,205	2,093,686	399.1	745,237	745,237	82.7	101,640	101,640	11.3	35,892
Tennessee	4,192,460	2,342,026	2,331,290	96.8	1,206,932	1,206,932	38.4	492,461	492,461	13.4	161,777
Texas	11,989,350	12,124,223	11,110,121	1,083.1	827,641	727,915	23.8	76,061	60,474	6.0	90,840
Utah	2,067,154	1,677,446	1,517,840	171.0	458,762	427,398	26.1	19,921	19,921	.9	101,995
Vermont	924,306	829,040	729,663	20.7	239,160	180,517	2.5				14,126
Virginia	3,652,667	2,926,358	2,865,020	933.0	526,731	514,550	100.0	52,053	52,053	.8	221,044
Washington	3,026,161	3,088,032	2,776,368	162.6	294,844	223,272	1.8				26,521
West Virginia	2,231,412	849,360	845,863	38.5	1,473,870	1,337,971	56.4	57,960	47,577	5.4	
Wisconsin	4,823,884	4,898,678	4,426,150	329.2	470,612	348,912	14.3	24,572	24,204	.3	24,617
Wyoming	2,219,155	1,655,804	1,653,365	124.1	549,869	546,972	28.4	22,200	18,609		209
District of Columbia	949,496	949,496	949,496	8.8							
Hawaii	926,033	635,899	617,792	8.9	322,161	298,335	8.5				9,906
TOTALS	195,000,000	140,421,321	134,060,449	10,832.3	48,056,728	45,389,109	1,957.0	8,478,098	7,202,522	313.0	8,347,920

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MAY 31, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 2,686,501	$ 2,686,501	37	1	9	$ 1,152,869	$ 1,152,869	11			$ 87,220	$ 87,220	1		3	$ 108,027
Arizona	1,256,099	1,119,207	1,094,094	13			104,506	104,506	1			9,740	9,740			3	47,759
Arkansas	3,574,060	1,791,281	1,785,984	38	4	2	1,558,421	1,556,271	17	2		199,984	199,714	1		30	32,092
California	7,486,362	5,041,619	4,875,772	29	8		2,366,957	2,298,880	17			311,710	205,938	6			105,772
Colorado	2,631,567	1,234,005	1,200,507	21	1		939,941	939,940	8			75,890	75,890	1			415,231
Connecticut	1,712,684	73,479	73,479		1		779,991	758,630	5		1	283,610	283,610	2			596,965
Delaware	418,239	130,000	130,000	1								277,993	277,993	2			10,246
Florida	2,827,883	1,660,162	1,657,852	17	5		826,541	826,197	12	1		79,200	79,200	1			264,634
Georgia	4,895,949	59,085	57,593	1	2		905,444	905,444	19	4		493,098	493,098	12		44	3,439,814
Idaho	1,674,479	908,936	902,471	14	1	5	385,595	385,482	5	1	9	248,737	248,737	2		7	137,789
Illinois	10,307,184	4,846,297	4,842,579	49	2		5,305,219	5,305,219	27	5							159,386
Indiana	5,111,096	1,830,400	1,694,405	20	11		3,427,682	3,391,082	22	1	162						25,609
Iowa	5,600,679	2,784,798	2,720,038	68	7	4	2,681,187	2,664,571	37	2	2	229,876	216,070	2		2	
Kansas	5,246,258	2,384,404	2,372,609	36		4	2,914,248	2,839,405	22	1	1						34,244
Kentucky	3,672,387	1,051,182	1,050,873	13	3		1,817,182	1,527,450	9	3		814,500	814,500	4	5		279,564
Louisiana	3,213,467	288,468	288,468	4			1,681,522	1,681,522	16	2		1,209,330	1,023,934	9	3		219,543
Maine	1,426,861	798,899	797,486	14		2	347,194	346,603	6	1		240,350	240,350	1			42,422
Maryland	2,061,751	358,388	358,388	3	3		518,504	518,504	3		19	638,004	615,536	3	2		569,323
Massachusetts	4,210,833	1,001,729	1,001,643	8	2		2,033,499	2,033,499	15	2		1,059,621	1,056,041	4			119,650
Michigan	6,765,197	3,476,224	3,394,989	35	4		3,397,177	3,326,708	10	4		43,500	43,500		1		
Minnesota	5,395,441	3,161,230	3,091,181	63	10	37	2,290,188	2,248,093	22	3	3	3,448	3,448			1	52,719
Mississippi	3,241,475	1,172,764	1,172,600	31	3	1	1,481,858	1,481,858	24	2	1	105,000	105,000	2		10	482,017
Missouri	6,142,153	786,713	783,740	14			5,560,303	5,250,107	35	1	1	1,650	1,650			1	106,655
Montana	2,722,327	2,538,405	2,494,780	35	7		178,582	178,582	2			75,278	30,000		1		18,965
Nebraska	3,556,441	1,939,514	1,929,459	65	2	8	1,382,819	1,382,819	15	1	3	31,678	31,678			13	212,485
Nevada	887,260	692,129	690,197	7	3		211,658	193,432	1			3,630	3,630			5	1
New Hampshire	822,484	341,748	341,748	3	3		395,145	395,100	5	1	1	15,452	15,452			6	70,184
New Jersey	3,983,826	760,036	760,036	6	2		2,558,322	2,547,277	15	5		139,100	139,100	1			537,414
New Mexico	1,725,286	1,044,613	1,043,684	13	1		673,090	670,400	5								11,202
New York	13,577,189	3,340,870	3,328,645	12	16		9,694,911	9,446,265	31	31		388,150	388,150		1		414,129
North Carolina	4,823,958	1,777,842	1,777,842	23	15		2,186,507	2,166,956	25	3	108	63,890	63,890	1			815,270
North Dakota	3,207,473	1,043,506	1,043,216	23	1		1,775,043	1,771,763	28	3							392,494
Ohio	8,439,897	347,806	282,313	2	1	2	5,595,268	5,438,779	38	6	1	2,625,300	2,301,242	16	1		417,562
Oklahoma	5,004,711	2,676,501	2,670,355	42	4	1	970,588	970,588	14	3	4	873,383	873,383	8	2	31	490,384
Oregon	2,334,204	1,193,881	1,187,909	10	5	2	1,170,816	1,089,215	6	1		49,093	49,093	1			7,987
Pennsylvania	11,483,613	2,256,542	2,009,765	37	9	9	8,086,045	7,594,504	39	11		1,079,528	968,128	7			911,216
Rhode Island	699,691	624,125	623,059	4	1		61,626	61,626		2							15,006
South Carolina	3,059,956	991,114	980,397	20	5	3	1,230,084	1,221,671	23	6	8	278,450	275,700		3	37	582,188
South Dakota	3,249,086	1,042,381	1,042,175	27	2	25	1,686,171	1,686,171	35	3		203,010	203,010	3		25	317,730
Tennessee	3,903,979	754,800	750,456	15	2	18	2,339,790	2,339,790	25	1	4	490,020	490,020	6		14	323,713
Texas	10,855,982	6,539,708	6,531,664	99	13	21	3,460,579	3,460,572	30	1	10	543,156	543,155	1	1	148	320,591
Utah	1,230,763	338,576	337,171	3	1	1	873,921	873,921	14								19,670
Vermont	729,857	500,765	495,086	7	5	15	271,833	231,750	3	2	5						3,021
Virginia	3,774,287	1,977,625	1,867,760	33	11	8	989,658	989,658	11	6	3	362,584	356,795	5	1	9	560,074
Washington	3,095,041	2,119,054	2,094,014	19	10	8	1,026,083	990,668	4	2		10,359	10,359			3	
West Virginia	2,677,937	79,454	79,454	1		4	1,631,021	1,629,053	15	3		575,069	575,069	8		1	394,362
Wisconsin	5,022,683	2,601,047	2,559,272	27	4		2,380,496	2,289,244	10	3							174,167
Wyoming	1,360,841	653,621	653,513	9			538,776	538,775	3			101,000	85,100	1			83,453
Dist. of Columbia	410,804	170,643	170,643	2			254,921	226,161	1								14,000
Hawaii	453,703	170,404	170,389	2			351,976	283,314	3								
TOTALS	196,000,000	77,162,481	75,948,254	1075	191	189	94,451,757	92,210,894	744	129	346	14,320,591	13,484,123	111	21	393	14,356,729

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Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF MAY 31, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 15,673,675	$ 8,301,315	$ 3,813,459	770.8	$ 171,795	$ 55,923	$ 115,872	5.8		$ 285,822	1.5	$ 12,895	$ 44,689
Arizona	5,211,960	2,641,935	9,006,025	5,203,675	2,607,439	542.9	44,046	5,574	24,073	.1		810		2,711	9,613
Arkansas	6,748,335	3,428,049	10,979,516	6,637,864	3,371,459	622.4	120,291	70,019	49,922	3.9	$ 33,944	4,200	10.9	6,509	2,468
California	15,607,354	7,932,206	30,662,474	15,607,354	7,752,490	763.9	114,733		109,877						69,839
Colorado	6,874,530	3,486,006	11,269,838	6,874,530	3,465,039	633.2						10,880			10,087
Connecticut	2,865,740	1,454,868	4,559,316	2,794,586	1,319,700	74.0	23,300	23,300				36,690	.3	47,854	98,477
Delaware	1,819,088	923,395	2,782,832	1,818,804	916,149	128.9						7,100		284	145
Florida	5,231,834	2,661,343	9,024,260	5,175,534	2,414,688	307.3	289,764	56,300	233,464	2.9					13,190
Georgia	10,091,185	5,113,491	13,462,239	9,272,425	3,313,943	766.1	1,056,956	375,323	681,633	65.7	84,072	240,400	10.2	359,365	877,516
Idaho	4,486,249	2,277,486	7,078,041	4,486,249	2,154,127	501.4	87,853		85,693	.1		12,110			25,556
Illinois	17,570,770	8,921,401	26,474,576	17,337,114	7,948,338	724.2	1,141,513	129,245	885,007	13.3		7,100		104,411	80,956
Indiana	10,037,843	5,088,963	15,555,334	9,892,557	4,814,846	482.8	227,844	129,083	98,761	2.4		61,727		16,203	113,629
Iowa	10,055,660	5,118,361	15,580,604	10,055,660	4,920,929	1,222.3	209,734		197,432	5.0					
Kansas	10,089,604	5,117,675	15,426,435	10,089,029	5,009,852	1,132.3	115,038		86,599	13.1				575	21,225
Kentucky	7,517,359	3,818,311	12,168,487	7,480,532	3,719,296	812.7	67,504		67,504	2.8	11,054	23,834		25,713	7,678
Louisiana	5,828,591	2,963,932	9,135,047	5,763,651	2,694,156	258.9	215,480		215,459	11.5		20,500	6.9	64,940	33,817
Maine	3,369,917	1,711,586	5,248,678	3,348,968	1,671,457	193.5	43,100	15,000	26,640	1.4		4,800	.1	5,949	8,790
Maryland	3,564,527	1,810,658	5,721,749	3,488,916	1,102,724	152.3	68,343		68,343	1.5		299,044	7.3	75,611	339,947
Massachusetts	6,597,100	3,350,474	10,401,583	6,550,778	3,094,281	115.5						91,944	.1	46,323	164,249
Michigan	12,736,227	6,452,568	20,726,161	12,736,227	6,298,778	768.1	96,788		96,788			40,000	.4		17,002
Minnesota	10,656,569	5,425,551	16,203,120	10,510,789	4,846,656	1,641.9	556,215	58,110	469,579	7.4		34,800	.6	87,670	74,516
Mississippi	6,978,675	3,540,227	12,796,784	6,795,654	3,000,647	724.4	628,159	125,000	488,813	21.2	3,960	14,860	.2	54,062	35,907
Missouri	12,180,306	6,173,740	18,185,313	12,088,271	4,998,938	1,441.2	1,125,699		1,121,765	6.3				92,035	53,036
Montana	7,439,748	3,769,734	11,764,611	7,425,285	3,655,874	1,058.4	56,546		56,546					14,463	57,314
Nebraska	7,828,961	3,964,364	12,969,984	7,812,918	3,746,021	1,035.5	201,659	16,043	185,616	16.3					32,727
Nevada	4,545,917	2,302,356	7,068,886	4,545,917	2,269,237	758.8	21,979		21,979			1,650			9,491
New Hampshire	1,909,839	969,462	3,007,886	1,904,951	953,689	78.3								4,888	15,773
New Jersey	6,346,039	3,220,879	9,213,234	6,166,170	2,448,230	88.4	1,197,850	100,280	663,430	8.6		9,212		79,589	100,007
New Mexico	5,792,935	2,941,700	8,908,408	5,749,069	2,920,579	749.8	3,536		3,536	.2		2,709		43,866	14,876
New York	22,330,101	11,327,921	39,994,755	22,196,547	10,551,869	823.7	806,142	119,856	525,499	1.6	6,600	112,200		7,098	138,353
North Carolina	9,522,293	4,840,941	14,951,839	9,205,888	4,513,514	1,345.2	564,251	303,110	261,141	21.4		6,400	.2	13,294	59,886
North Dakota	5,804,448	2,938,967	8,629,934	5,609,071	2,243,105	2,118.8	282,009	158,643	123,366	21.0		369,590	30.6	36,735	202,905
Ohio	15,484,592	7,865,012	24,759,236	15,377,745	7,513,301	792.1	404,871	98,678	245,354	6.7		45,000	1.5	8,170	61,356
Oklahoma	9,216,798	4,685,180	14,746,797	9,214,657	4,429,653	806.6	196,489		196,489	.4		36,174		2,140	22,864
Oregon	6,106,896	3,097,814	9,973,039	6,047,835	2,960,906	468.0	64,223		64,223	.8	55,000		.7	4,061	72,686
Pennsylvania	18,891,004	9,590,788	29,038,106	18,555,365	8,909,961	1,054.0	913,639	281,923	587,195	16.7		82,607	1.4	53,716	11,025
Rhode Island	1,998,708	1,014,572	3,150,270	1,998,708	1,012,094	89.2	2,478		2,478						
South Carolina	5,459,165	2,770,954	8,035,990	5,220,362	2,482,798	622.3	400,309	148,148	224,531	12.6	40,000	40,613	1.4	50,655	23,012
South Dakota	6,011,479	3,047,643	9,392,065	5,909,991	2,904,076	1,589.5	96,671	5,440	91,231	17.3	76,862	23,771	21.7	19,186	28,565
Tennessee	8,492,619	4,302,991	13,732,614	8,490,706	4,184,693	492.8	83,140		61,140	3.5	76	57,158	8.0	1,837	
Texas	24,244,024	12,291,253	38,019,184	24,197,558	11,908,921	2,781.9	360,325	2,797	347,542	.9		500	7.1	43,668	34,290
Utah	4,194,708	2,132,691	7,461,119	4,194,708	2,132,691	590.9									
Vermont	1,867,573	948,007	3,166,359	1,867,452	940,847	141.0								121	7,160
Virginia	7,416,757	3,765,387	11,591,713	7,328,534	3,347,391	621.2	268,951	27,651	235,668	28.8	6,735	9,652	.1	53,836	172,676
Washington	6,115,867	3,106,412	9,412,616	6,112,042	3,035,027	303.0	46,596		46,596					3,825	24,789
West Virginia	4,474,234	2,280,335	6,483,744	4,334,882	1,805,838	213.2	357,457	54,892	279,519	8.9		66,425	.7	84,460	128,553
Wisconsin	9,724,881	4,941,837	15,454,144	9,724,881	4,884,329	619.7	15,050		15,050						42,458
Wyoming	4,501,327	2,287,712	6,924,304	4,476,692	2,212,462	1,040.2	11,100	11,100		.3				13,534	75,250
District of Columbia	1,918,469	973,842	2,887,584	1,918,469	968,979	22.3						4,863			
Hawaii	1,871,062	949,778	2,972,062	1,857,812	555,221	53.7	359,759		342,210	2.2	13,250	39,680			12,667
TOTALS	394,000,000	200,000,000	631,832,670	389,754,757	184,740,697	35,139.5	13,119,185	2,371,438	9,703,463	332.6	331,553	2,104,825	111.9	1,542,252	3,451,015

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, No. 5 JULY 1937

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 5 July 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

LABORATORY, EXPOSURE, AND SIMULATED SERVICE TESTS OF SLOW-CURING LIQUID ASPHALTS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by R. H. LEWIS, Associate Chemist

and

W. O'B. HILLMAN, Assistant Highway Engineer

AN INVESTIGATION of the weather-resistant properties of a number of liquid asphalts of the slow-curing type was recently completed by the Bureau of Public Roads in cooperation with Committee 3–b on Cutbacks and Liquid Residuals of the Association of Asphalt Paving Technologists. A general report [1] on this work was given at the January 1936 meeting of the Association.

The data presented in that report were discussed only briefly and data on some phases of the investigation were omitted entirely. It is felt that the work warrants a more detailed discussion and therefore this report, which presents the viewpoint of the Bureau only, has been prepared.

The purpose of the investigation was to determine if the changes that occur in the physical and chemical properties of liquid asphalts present as binders in road-surfacing mixtures of the densely graded type and the behavior of the road mixtures under simulated service conditions could be correlated with the results of conventional laboratory tests on these products and with the results of exposure tests [2 3] similar to those previously made by the Bureau on liquid asphalts of the same type.

The investigation was carried out along the following lines:

1. The liquid asphaltic materials were examined by the test methods recommended by the Bureau of Public Roads and The Asphalt Institute, and by other test methods that have been or are in use in specifications for liquid asphaltic road materials of the slow-curing type.

2. The asphaltic materials were exposed in comparatively thin films to the action of solar light and heat and of air under different test conditions. The amount of volatile matter lost under these conditions was determined and the residues formed during the various periods of exposure were tested for consistency, ductility, and solubility.

3. Mixtures of the liquid asphalts with a sand and limestone-dust aggregate were made. Portions of these mixtures were molded into specimens for the Hubbard-Field stability test and for the toughness test (resistance to impact). The specimens were tested immediately, and after exposure to various test conditions for different periods of time.

4. The greater portions of these prepared mixtures were placed as wearing surfaces on a small circular track and subjected to weather conditions and controlled traffic of rubber-tired wheels. The behavior of the surfaces was noted and the different sections were cored from time to time to determine the changes in stability of the mixtures.

[1] Proceedings of the Technical Sessions of the Association of Asphalt Paving Technologists, Cleveland, Ohio, January 23, 1936.

[2] A Study of Some Liquid Asphaltic Materials of the Slow-Curing Type, by R. H. Lewis and W. O'B. Hillman. PUBLIC ROADS, vol. 15, no. 4, June 1934.

[3] Further Studies of Liquid Asphaltic Road Materials, by R. H. Lewis and W. O'B. Hillman. PUBLIC ROADS, vol. 16, no. 6, August 1935.

152397—37——1

PHYSICAL AND CHEMICAL PROPERTIES OF ASPHALTIC MATERIALS DIFFERED WIDELY

Slow-curing liquid asphaltic materials of the SC–2 type, similar to those investigated in previous studies by the Bureau, and having Furol viscosities of 250–320 seconds at 122° F., were selected for investigation. Information relative to the source of the base petroleum and the refining process used in the manufacture of the selected materials is as follows:

Sample 1: An uncracked, steam-reduced Mexican residual (probably 250–300 penetration asphalt) blended with gas oil to reduce the viscosity.

Sample 2: An uncracked, steam-reduced midcontinent residual.

Sample 3: An uncracked, steam-reduced California residual.

Sample 4: A 1,000-pound pressure cracking coil residual produced from gas oil.

Sample 5: A low-level residue, reduced from west Texas crude by the Dubbs process, blended with medium-heavy gas oil to reduce the viscosity. The transfer temperature of the Dubbs process was 910° F., chamber pressure 200 pounds, and flashed in a vacuum to approximately 4,000 Furol viscosity at 122° F.

Sample 6: A blend of 50 percent of sample 2 and 50 percent of sample 4.

The data obtained on sample 6 were not included in the report presented by committee 3–b. There was considerable difference of opinion among the members of the committee as to the advisability of including a blend of cracked and uncracked products in this study. It was held that the difference in behavior of uncracked materials, as represented by samples 1, 2, and 3, and of cracked materials, as represented by samples 4 and 5, was of major importance. The behavior of a blend of two materials in only one proportion would not give an accurate picture of the probable behavior of blends of other materials or blends of the same materials in different proportions. The committee therefore agreed that the behavior of blends should be considered in a separate study. The behavior of sample 6 under the various test conditions, however, was of such unusual character that the Bureau believes the data relative to this material should be included in this report.

Results of the laboratory tests on the liquid asphalts, together with the test requirements for SC–2 grade material, are given in table 1. It will be seen from a study of this table that, with the exception of sample 1,

TABLE 1.—*Results of tests on original materials showing conformity with the requirements for the SC–2 grade of liquid asphaltic material*

	Sample no.						Requirement for SC–2 grade
	1	2	3	4	5	6	
Specific gravity at 77° F.	0.970	0.942	0.972	1.054	1.010	1.000	.....
Flash point, °F.	220	500	295	380	270	420	200+
Furol viscosity at 122° F. seconds	315	291	312	272	256	283	200–320
Float at 77° F. do	19	100	23	65	37	61	.....
Loss at 325° F., 5 hr., 50 g. percent	12.3	0.2	5.7	1.1	5.4	0.7	.....
Loss at 325° F., 5 hr., 20 g. do	19.4	0.2	10.7	1.8	9.2	1.1	.....
Reaction to Oliensis test	Negative	Negative	Negative	Positive	Positive	Positive	.....
Organic matter insoluble in CS_2 percent	.....	.....	.....	0.18	0.38	0.10	.....
Organic matter insoluble in CCl_4 do	0.09	0.04	0.06	0.39	2.24	0.15	.....
Organic matter insoluble in 86° B. naphtha do	18.3	3.0	5.4	11.2	16.9	6.7	.....
Fixed carbon do	9.0	3.0	3.7	7.2	10.8	5.1	.....
Asphaltic residue do	67.1	72.1	62.6	64.5	74.4	54.6	.....
Distillation (A. S. T. M. D402–34T):							
Total amount off by volume at 437° F. do	.....	.....	.....	.....	.....	.....	2–
Total amount off by volume at 600° F. do	[1] 15.5	.....	.....	.....	1.5	.....	15–
Total amount off by volume at 680° F. do	[1] 25.5	0	11.0	0	15.0	0	25–
Total distillate to 680° F. percent (by weight)	22.0	0	9.8	0	12.3	0	.....
Distillation loss [2] do	4.7	2.1	5.0	4.6	4.7	3.0	.....
Total loss in distillation test do	26.7	2.1	14.8	4.6	17.0	3.0	.....
Tests on the distillation residue:							
Float at 122° F. seconds	196	40	41	29	89	29	25+
Organic matter insoluble in CS_2 percent	.....	.....	.....	0.19	0.45	0.10	1.0–
Organic matter insoluble in CCl_4 do	0.09	0.09	0.13	0.46	1.61	0.07	.....
Organic matter insoluble in 86° B. naphtha do	25.1	3.3	6.1	12.1	22.0	7.3	.....

[1] Failed to meet specification.

[2] Loss during the pouring and cooling of the residue.

which had percentages of distillate at 600° F. and 680° F. that were 0.5 percent higher than the specification limit, all of the materials met the SC–2 specification. The test data do indicate, however, that although these materials met the same specification their physical and chemical properties differed widely. These differences will be discussed briefly.

The three steam-reduced products, samples, 1, 2, and 3, had lower specific gravities than did samples 4, 5, and 6 and both the original materials and their distillation residues were homogeneous in the Oliensis test. When viewed under the microscope at 200 magnifications, samples 1 and 3 had clear fields both when undiluted and when diluted with 6 parts of carbon tetrachloride. Sample 2 showed a few carbonaceous particles under both conditions. The two cracked products, samples 4 and 5, and the blend, sample 6, had the highest specific gravities, were heterogeneous in the Oliensis test, and all contained a number of carbonaceous flecks when viewed microscopically.

Photographs of the Oliensis stains are given in figure 1, and photomicrographs of samples 2, 4, 5, and 6 are shown in figure 2.

All of the products except sample 5 were almost completely soluble in carbon disulphide, and sample 5 had less than 0.5 percent of insoluble matter. However, when carbon tetrachloride was used as the solvent, the solubilities of samples 4 and 5 were considerably decreased, showing the presence of carbenes in these two cracked materials. The percentage of material insoluble in 86° B. naphtha ranged from a maximum of 18.3 for sample 1 (the Mexican base material), to a minimum of 3.0 for sample 2 (the midcontinent residual). The two cracking coil residuals both contained less material insoluble in naphtha than did the steam-reduced Mexican residual.

BLENDED MATERIAL SHOWN TO BE UNSATISFACTORY IN ASPHALTIC RESIDUE TEST

The distillation curves shown in figure 3 illustrate the wide range in volatile matter present in samples 1, 3, and 5. Since gas oil was used as the diluent to reduce the viscosity of the base materials of samples 1 and 5 to specification limits, the high distillation losses undoubtedly resulted from the use of this cutting medium by the producer. As indicated by the total loss by weight in the distillation test, which includes the loss on cooling and pouring, sample 1 had the greatest amount of volatile constituents. Although no distillate was actually recovered from samples 2, 4, and 6, the distillation loss or loss on pouring and cooling of the residue to room temperature served to classify sample 2 as the least volatile material.

When the consistency of the distillation residue is considered in connection with the amount of volatile matter, the great difference in the hardening properties of these products is evident. The low amounts of volatile matter contained in samples 2, 4, and 6 indicate that they would be extremely slow hardening. Sample 3, although having a relatively high distillation loss, developed a residue with the same float-test consistency as sample 2. Samples 1 and 5 both had high percentages of distillation loss and the high float-test values for their distillation residues indicated that they were the most rapid hardening of the six materials.

The results of the volatilization tests on 20- and 50-gram samples (table 2) show in the same manner that sample 1 had the most volatile matter and was the most rapid hardening. Samples 2, 4, and 6 had the least volatile matter and were indicated by the test results to be the slowest-hardening materials. Although in the distillation test sample 5 lost more than sample 3, in the volatilization test sample 3 lost more than sample 5. However, in both tests sample 5 produced a slightly more viscous residue as measured by the float test.

A comparison of the solubilities of the residues from the distillation and volatilization tests showed that, except for the solubility in 86° B. naphtha of sample 5, the residues from volatilization were less soluble in carbon disulphide, carbon tetrachloride, and naphtha than were the residues from distillation, although in every case the samples lost more in the distillation test. The results of the solubility tests indicate that the volatilization test caused greater inherent alterations than did the distillation test.

The results of the asphaltic residue test are given in table 3. One residue was obtained by evaporating the materials at a temperature of 480° to 500° F., the con-

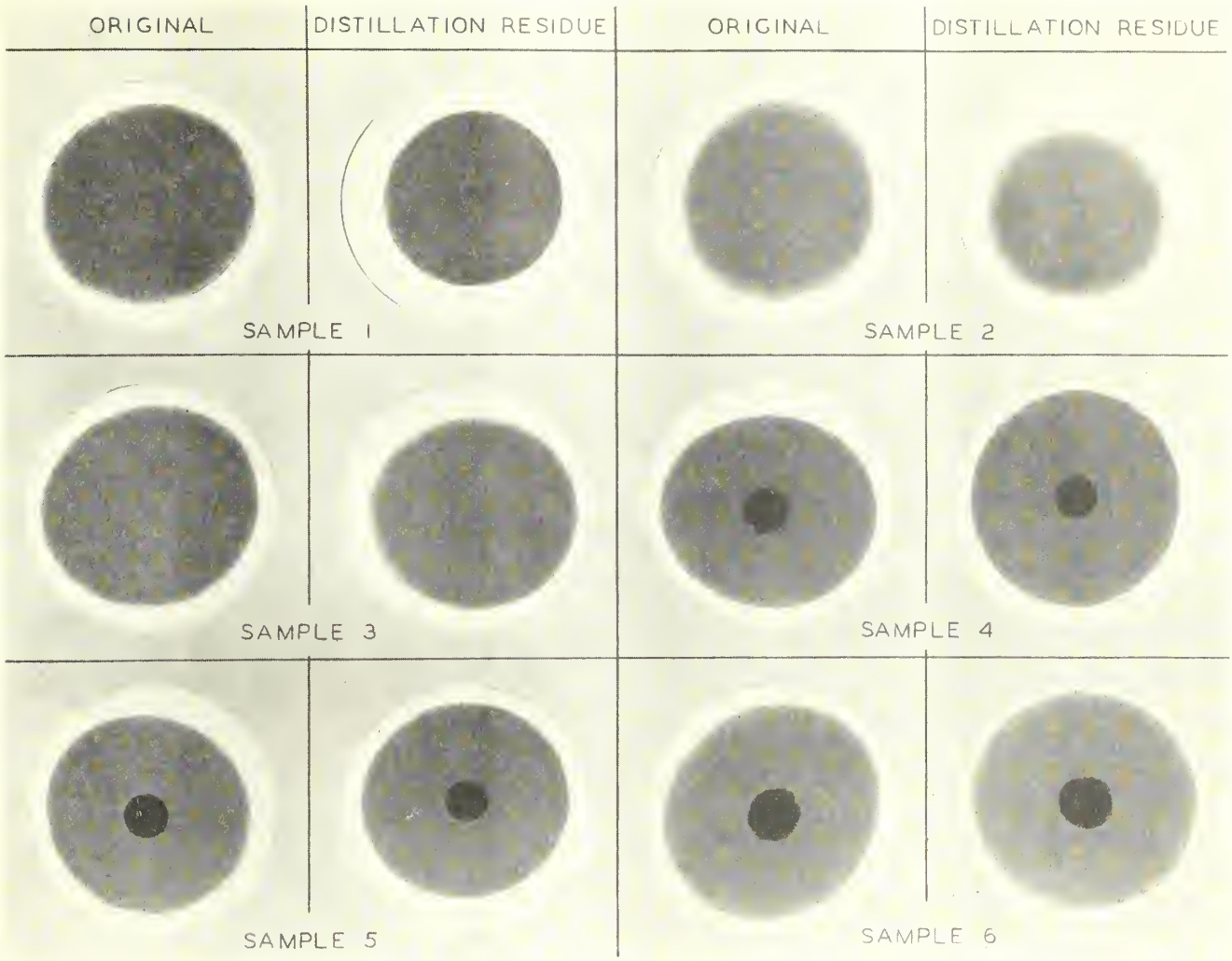

FIGURE 1.—APPEARANCES OF OLIENSIS SPOTS OF ORIGINAL MATERIALS AND THEIR DISTILLATION RESIDUES.

TABLE 2.—*Results of volatilization tests*

Sample	1	2	3	4	5	6
Loss at 325° F., 5 hr., 50 g ..... percent	12.3	0.2	5.7	1.1	5.4	0.7
Float of residue at 122° F ..... seconds	47	22	26	25	31	20
Loss at 325° F., 5 hr., 20 g ..... percent	19.4	0.2	10.7	1.8	9.2	1.1
Tests on residue of 20-gram sample:						
Float at 122° F ..... seconds	125	27	39	29	64	22
Organic matter insoluble in CS_2 ..... percent	.....	.....	.....	0.21	0.94	0.23
Organic matter insoluble in CCl_4 ..... percent	0.10	0.10	0.23	0.76	2.59	0.41
Organic matter insoluble in 86° B. naphtha ..... percent	33.7	3.9	8.4	16.1	21.6	9.0

trol temperature of the standard test method. The other residue was obtained by evaporating at a temperature of 400° to 420° F. Although sample 6 lost but little in the distillation and volatilization tests, the residues obtained were smooth and of even texture and there were no indications that the blend would become incompatible. However, when sample 6 was evaporated at a temperature of 480° to 500° F. to a penetration of 100, the residue became granular in texture and gravish black in color, and when tested had a high softening point and very little ductility.

It is of interest to note that sample 6, a combination of 50 percent of sample 2 which had an asphaltic residue of 72.1 percent and 50 percent of sample 4 which had an asphaltic residue of 64.5 percent, had only 54.6 percent of asphaltic residue of 100 penetration. When an attempt was made to reduce the sample at the lower temperature (400°–420° F.) it separated into two parts, one soft and greasy, the other hard and granular. These could not be refluxed and reduction to a satisfactory residue was impossible.

TEST RESULTS SHOW NON-ASPHALTIC CHARACTER OF SAMPLE 2

Of the other materials, sample 5 was most rapidly reduced to the desired penetration, followed closely by sample 1; samples 4 and 3 were next; and sample 2 took an exceedingly long time to be reduced. At the lower temperature the time of reduction for all samples was appreciably increased, and the percentage of residue obtained was in every case greater than that obtained under the standard test. The increase in percentage of residue ranged from 2.3 percent for sample 1 to 11.5 percent for sample 2 with an average increase for the five samples of 5.2 percent.

Under both temperatures of reduction samples 1, 3, 4, and 5 produced residues having high ductility at 77° F., and both residues of sample 1 had good ductility at 34°–35° F. The residues of samples 3 and 5 obtained at the higher test temperature had no ductility at 34°–35° F. The residue of sample 3 obtained at the lower temperature had a ductility of 5.5 centimeters at 34°–35° F. and the residue of sample 5, although it

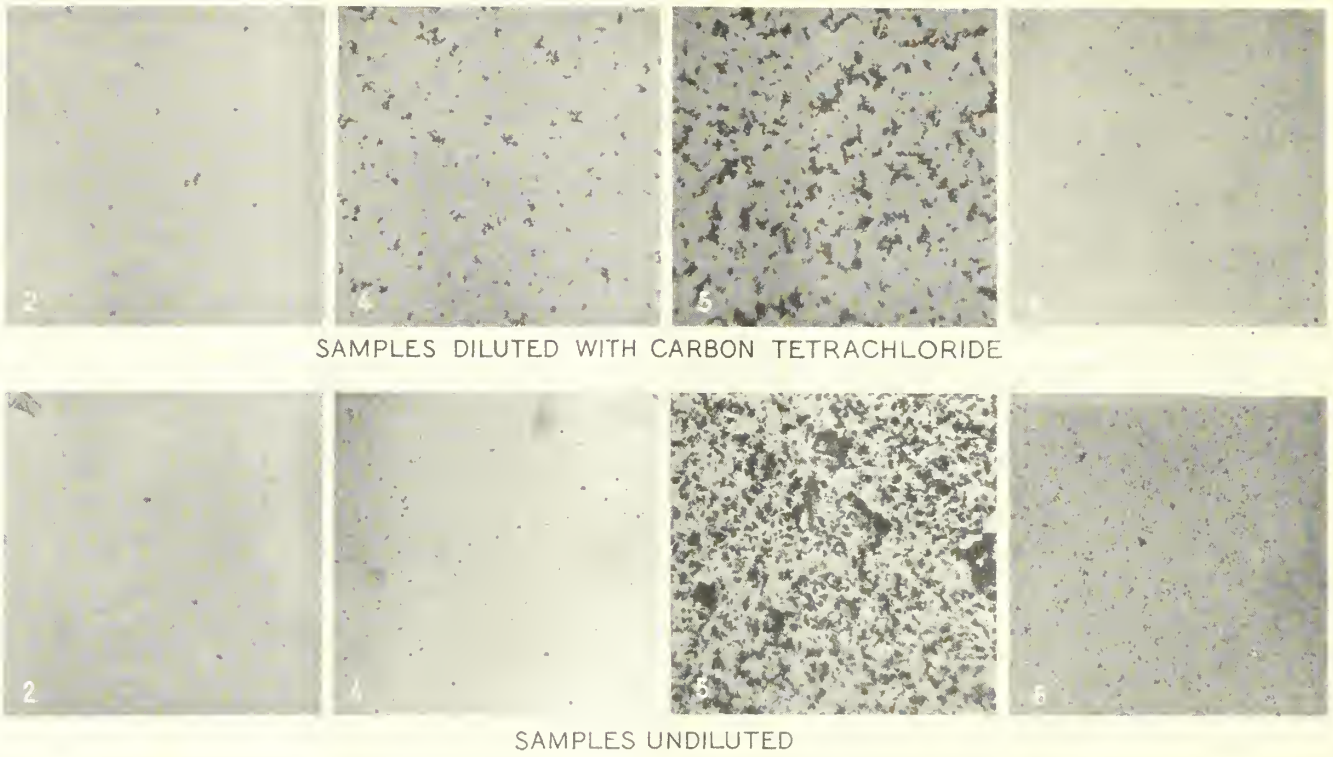

FIGURE 2.—PHOTOMICROGRAPHS OF DILUTED AND UNDILUTED ASPHALTIC MATERIALS. MAGNIFIED 115 DIAMETERS.

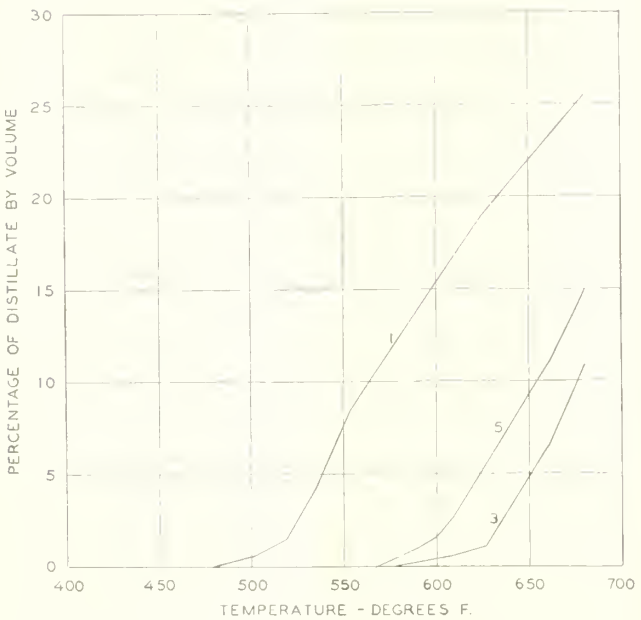

FIGURE 3.—DISTILLATION CURVES OF SAMPLES 1, 3, AND 5.

did not pull to a thread, had a ductility of 3.0 centimeters. Both residues of sample 4 had no ductility at 34°–35° F.

Both of the asphaltic residues of samples 2 had low ductility at 77° F. but retained a high percentage of their original ductility when tested at 34°–35° F. The high penetrations obtained on the residues of this sample at 32° F., together with their high softening points, indicate that these residues were the least susceptible to temperature change. The residues were greasy, however, and appeared much harder than the penetration test indicated. The character of the asphaltic residues, the long time of reduction to the desired penetration, together with the low amount of material insoluble in naphtha in the original material, show that the base petroleum was essentially a paraffin-base oil.

Although viscosity limits in specifications for liquid bituminous materials only insure the use of products of comparable workability and initial consistency, viscosity tests made at various temperatures are of value in classifying various types of material meeting the same specification. Viscosity tests made at various temperatures also can be used to distinguish different types of material according to their susceptibility to temperature change. Accordingly, the Furol viscosities of the six samples under investigation were determined at 104°, 122°, 140°, 158°, and 180° F.

When the resulting viscosities are plotted against the temperature on logarithmic scales (fig. 4) the points for each sample fall generally along a straight line. The equation of this line is $\log V = -a \log t + K$, where V is the Furol viscosity in seconds, t is the temperature in degrees Fahrenheit, $-a$ is the slope of the curve, and K is the Y intercept. According to this equation K is also the logarithm of the viscosity at 1° F. The coefficient $-a$ may be used as a suseeptibility factor, the greater the numerical value of $-a$ the greater is the suseeptibility of the material. Applying this factor, it is seen that sample 4 was the most susceptible to changes in temperature, followed in order by samples 6, 3, 5, 2, and 1.

The float-test consistency of these materials as determined at 77° F. is of interest in demonstrating the unusual character of sample 2. A study of a large number of asphaltic, semiasphaltic, and cracking-coil residuals of the same consistency as the six materials used in this study showed that for materials of the same viscosity at 122° F. cracking-coil products, because of their greater susceptibility to temperature change, had the

TABLE 3.—*Results of asphaltic residue tests*

		Samples reduced at 480°–500° F						Samples reduced at 400°–420° F.[1]				
		1	2	3	4	5	6	1	2	3	4	5
Time of reduction	minutes	39	600	83	81	35	300	275	1,600	400	400	200
Residue	percent	67.1	72.1	62.6	64.5	74.4	54.6	69.4	83.6	66.7	69.3	78.0
Tests on residue:												
Penetration at 77° F., 100 grams, 5 seconds		86	99	110	109	95	103	104	100	104	83	106
Penetration at 32° F., 200 grams, 60 seconds		32	59	18	21	22	27	39	55	23	19	23
Softening point	°F.	122	126	109	106	111	139	121	139	111	111	111
Ductility at 77° F.	centimeters	90	11	110+	110+	110+	4.5	110+	4.5	110+	110+	110+
Ductility at 34–35° F.	centimeters	5.5	3.3	0	0	0	0	6.5	2.5	5.5	0	[2] 3.0
Organic matter insoluble in CS_2	percent				0.24	0.66	3.42				0.10	0.81
Organic matter insoluble in CCl_4	percent	0.10	0.14	0.04	1.33	2.60	9.33	0.09	0.11	0.09	1.94	2.56
Organic matter insoluble in 86° B. naphtha	percent	28.4	22.5	15.3	31.1	25.8	32.4	28.7	25.0	18.3	27.3	25.6

[1] Sample 6 separated into 2 parts which could not be refluxed.

[2] Sample broke, not a normal test.

higher float values when tested at 77° F. The high float of sample 2 when tested at 77° F. is, however, typical of results on nonasphaltic residuals of comparable viscosity.

MATERIALS EXPOSED IN VARIOUS FILM THICKNESSES FOR VARIOUS PERIODS

In exposing the materials to the action of sunlight, heat, and air, the test procedure followed that used in previous investigations by the Bureau. In this study, however, the materials were exposed not only in layers ⅛-inch thick but also in 1/16- and 1/32-inch layers. Three samples of each thickness were prepared of each material. Each sample, containing 50 cubic centimeters of material, was placed in a seamless, flat-bottom aluminum pan ⅝ inch deep and of sufficient diameter to give the desired film thickness. The samples were placed in the exposure boxes on June 6, 1934. One set was removed and tested at the end of 5 weeks, another at 10 weeks, and the third at 15 weeks. The losses of volatile matter were determined by weighing the samples at the end of 2, 12, 35, 70, and 105 days.

During the exposure period hourly temperature readings were made daily from 9 a. m. to 4 p. m. The maximum daily temperature within the boxes varied from 85° to 185° F. with an average daily maximum of 145° F. The maximum temperatures occurred on days with the maximum possible sunshine. On cloudy days there was little difference between the temperature inside and outside the boxes. United States Weather Bureau reports showed that the samples exposed for 5, 10, and 15 weeks were subjected to 368, 663, and 904 hours of sunlight, respectively. The appearances of the samples exposed in ⅛-inch films, after 15 weeks of exposure, are shown in figure 5. The results of tests on the residues are given in table 4.

The residues shown in figure 5 are briefly described as follows:

Sample 1: Rough, dull center, brown streak when scratched, some glossy material around edges in cracks.

Sample 2: Smooth, glossy, greasy, slightly mottled.

Sample 3: Smooth, glossy, sticky.

Sample 4: Smooth, glossy, slightly sticky.

Sample 5: Rough surface, raised portions dull, depressions slightly glossy.

Sample 6: Generally glossy, rough; cracks filled with soft material which was fluid and greasy.

Although not indicated in the table because of variations in the losses by individual specimens of the same sample, each individual specimen, with the exception of those of sample 2 which gained weight throughout the test, lost weight progressively as the time of exposure increased. The table shows that the greatest losses occurred in the thinnest films. The order of the losses at the end of 15 weeks is in substantial agreement with the order of the losses in the distillation and volatilization tests.

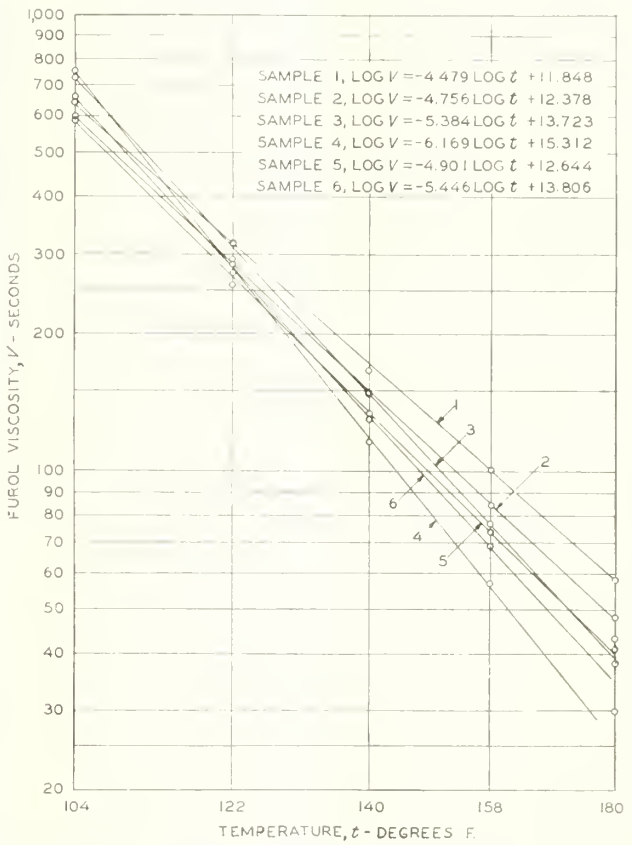

FIGURE 4.—RELATIONS BETWEEN FUROL VISCOSITY AND TEMPERATURE FOR THE VARIOUS SAMPLES.

In figure 6 the losses at various periods of exposure are plotted for samples 1, 3, 4, and 5. The losses at 2, 12, and 35 days are the averages for the three specimens of each sample originally exposed. The losses at 70 days are the averages for the materials tested at the end of 10 and 15 weeks, while the losses at 105 days are the losses for the specimens of each sample tested at the end of 15 weeks. This accounts for the fact that the ⅛-inch film of sample 3 appeared to have lost more in 10 weeks than in 15 weeks.

Sample 1 not only lost more than the other samples but lost at a faster rate. In 12 days sample 1 had lost, depending upon the film thickness, from 75 to 79 percent of its total loss in 15 weeks. In the same time

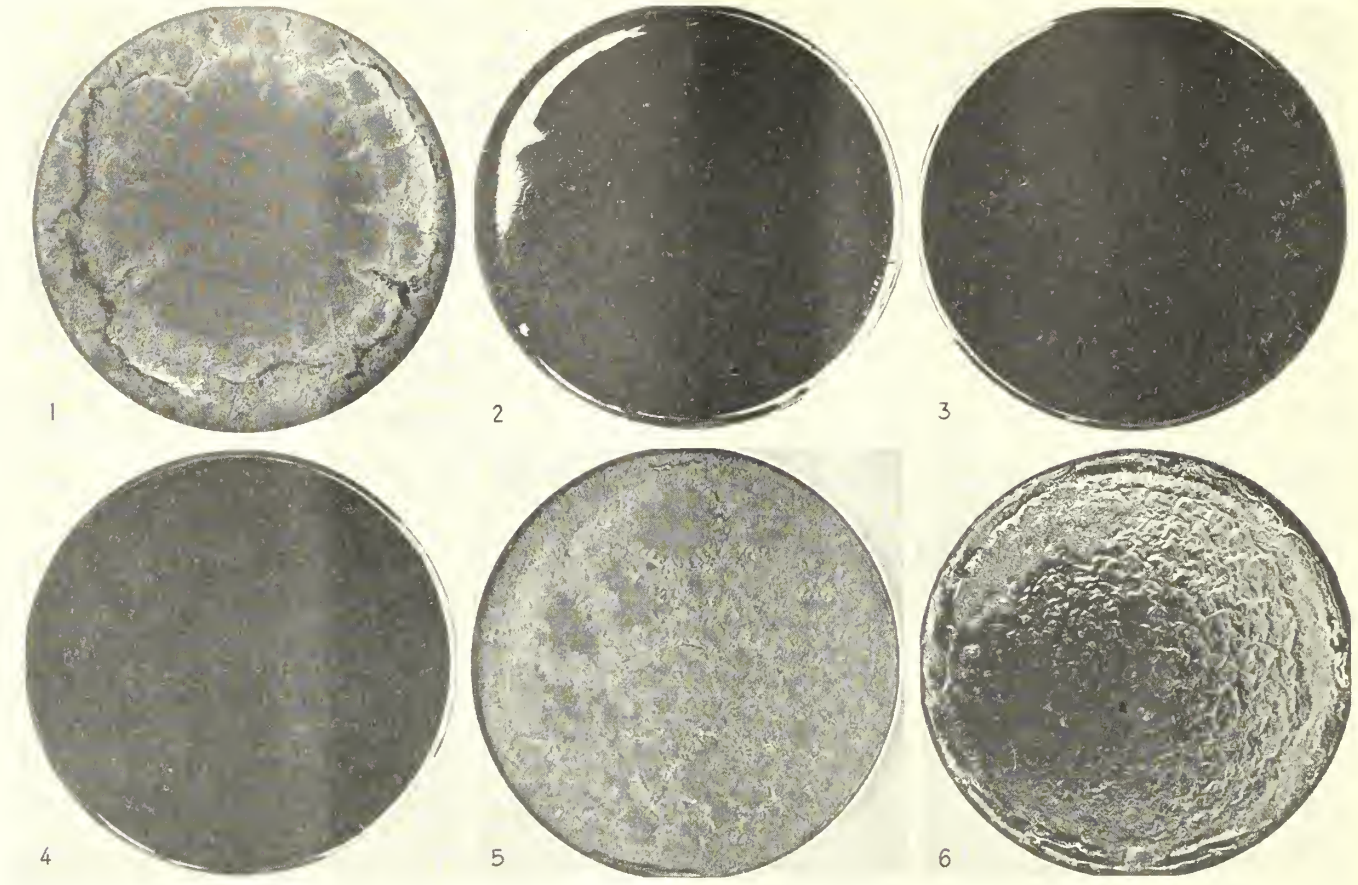

FIGURE 5.—APPEARANCES OF SURFACES OF THE $\frac{1}{8}$-INCH FILMS OF LIQUID ASPHALT AFTER 15 WEEKS OF EXPOSURE.

sample 5 had lost from 45 to 72 percent of its total loss; sample 3 lost from 43 to 56 percent; and sample 4 lost from 14 to 30 percent. In 35 days sample 1 had lost from 90 to 97 percent of its total loss; sample 5 lost from 89 to 91 percent; sample 3 lost from 80 to 86 percent; and sample 4 lost from 49 to 61 percent.

These comparisons show that, although the volatilization test gave an indication of the comparative volatility upon exposure, the distillation test gave a better idea of the probable rate of volatility. The effect of the gas oil used by the producer for reducing the base materials of samples 1 and 5 to the desired consistency was shown by the greater rate of volatility of these two materials under both laboratory and exposure conditions.

MATERIALS EXPOSED IN THINNEST FILMS MOST HIGHLY ALTERED

Within a week after the start of exposure, sample 6, separated into two parts, one soft and greasy, the other hard and granular. These two components could not be fluxed together and the tests on the residues were made after the samples had been mixed to as uniform a condition as possible. It will be noted that neither the distillation nor the volatilization tests gave any indication that this blend might prove to be an incompatible mixture. The granular residue obtained from the asphaltic residue test made at the standard temperature was indicative of some instability, and the behavior of the material in the asphaltic residue test, run at a temperature of 400°–420° F., was further evidence that this material might prove to be a noncohesive and unsatisfactory product.

Table 4 shows that the thinner films at any exposure period had the harder residues. Excluding sample 6, sample 2 was the slowest-hardening material. After 15 weeks the specimen of sample 2 exposed in a film one thirty-second inch thick had hardened to a residue having a penetration of 211. This residue was very greasy and unlike a normal asphaltic material of similar consistency. It had a float value greater than 1,200 seconds at 122° F., a high softening point for material of this penetration, and very little ductility at 77° F. All other exposure residues of sample 2 had penetrations over 300 and were also greasy.

The remaining 4 materials (samples 1, 3, 4, and 5) had hardened to a much greater extent. When exposed in $\frac{1}{8}$-inch films, sample 1 was the hardest after each of the three test periods, followed by samples 5, 3, and 4. When exposed in $\frac{1}{16}$-inch films, sample 1 was the hardest after each of the test periods, and at the end of 5 and 10 weeks samples 5, 3, and 4 followed in that order. However, after the last 5-week period sample 4 had increased in hardness until it was second to sample 1, followed by samples 5 and 3. When exposed in a $\frac{1}{32}$-inch film sample 5 was the hardest at the end of 5 weeks, followed in order by samples 1, 4, and 3. At the end of 10 weeks sample 1 was the hardest, followed by samples 4, 5, and 3, and at the end of 15 weeks sample 4 was the hardest, followed by samples 1 and 5 with the same penetration, and sample 3 was the softest, excluding samples 2 and 6.

For the thicker films and short periods of exposure the hardness of the residues was roughly proportional to the loss in weight. In the thinner films and for longer periods of exposure, the character of the material apparently influenced the hardening of the exposed residues and the cracked materials, samples 4 and 5, ultimately became the hardest.

TABLE 4.—*Results of tests on thin films of liquid asphalt exposed under glass*

Time of exposure	5 weeks (368 hours of sunlight)						10 weeks (663 hours of sunlight)						15 weeks (904 hours of sunlight)					
Sample no.	1	2	3	4	5	6[1]	1	2	3	4	5	6[1]	1	2	3	4	5	6[1]
Sample exposed in ⅛-inch film:																		
Loss, percent	22.4	−1.0	11.3	1.6	13.1	0.1	23.9	−1.1	15.1	2.8	15.1	0.1	23.2	−0.9	13.8	4.1	15.0	0.6
Float at 122° F., seconds		50	95	58		49		73	252	100		51		82	206			71
Penetration at 77° F., 100 grams, 5 seconds	59				140		38		171		60		41		205	147	55	
Softening point, ° F.	144	91	77	67	131		167	91	102	85	162		163	90	99	104	165	
Ductility at 77° F., centimeters	8.5				8.8		4.7		110+		2.8		5.5			110+	3.5	
Organic matter insoluble in CS_2, percent	0.21			0.83	0.92	0.35	0.46			1.80	1.48	0.58	0.66	0.14	0.11	2.87	1.89	1.48
Organic matter insoluble in CCl_4, do	0.14			2.54	4.49	1.70	0.53	0.10	0.09	4.76	5.63	2.29	0.75	0.07	0.13	6.21	6.45	5.09
Organic matter insoluble in 86° B. naphtha, percent	35.0	8.8	17.8	21.0	28.4	16.1	37.0	11.0	24.1	24.4	32.3	21.0	39.0	12.8	23.5	27.8	33.0	25.2
Sample exposed in 1/16-inch film:																		
Loss, do	23.1	−1.1	14.9	2.4	15.6	0.6	24.3	−1.3	17.8	4.3	16.8	0.7	25.5	−1.3	17.7	5.7	16.8	1.1
Float at 122° F., seconds		74	230	115		67		112				93		214				136
Penetration at 77° F., 100 grams, 5 seconds	40		192		59		25		52	55	36		18	300+	41	24	26	
Softening point, ° F.	161	90	100	87	175		190	98	127	122	215		226	114	132	149	215	
Ductility at 77° F., centimeters	5.0		110+		1.7		2.6		26.5	6.0	[2] 0.5		[2] 2.0		20	[2] 0.3	[2] 0.3	
Organic matter insolube in CS_2, percent	0.40			2.20	1.96	1.03	0.87			4.32	2.85	2.31	1.42	0.32	0.22	6.51	5.31	5.13
Organic matter insoluble in CCl_4, do	0.45			5.80	7.19	2.87	0.89	0.13	0.14	9.08	9.40	5.45	1.67	0.39	0.26	12.62	11.43	8.84
Organic matter insoluble in 86° B. naphtha, percent	37.8	11.8	23.1	25.3	34.0	22.9	38.7	16.0	31.4	30.6	38.5	34.5	45.1	19.3	32.2	34.7	41.0	35.5
Sample exposed in 1/32-inch film:																		
Loss, do	23.6	−1.4	16.5	4.6	16.8	1.3	25.5	−1.3	18.6	7.1	17.8	0.9	26.1	−1.4	20.1	7.9	18.4	1.5
Float at 122° F., seconds		134				129		1,200+				156		1,200+				1,200+
Penetration at 77° F., 100 grams, 5 seconds	35		76	57	29		10	300+	26	13	18		9	211	15	5	9	300+
Softening point, ° F.	188	102	120	124	205		219	126	145	173	226		247	131	175	202	256	
Ductility at 77° F., centimeters	3.5		55	11	[2] 0.3		[2] 1.0		4.5	0	0		0	3.0	[2] 1.0	0	0	
Organic matter insoluble in CS_2, percent	0.52	0.17	0.12	4.87	4.51	2.98	1.88	0.73	0.50	8.13	10.03	9.20	2.88	1.65	1.21	12.77	11.44	12.52
Organic matter insoluble in CCl_4, do	0.62	0.18	0.20	9.94	10.84	7.90	1.98	0.74	0.45	15.56	17.14	14.65	3.22	2.06	1.40	20.12	18.00	16.42
Organic matter insoluble in 86° B. naphtha, percent	41.3	15.8	27.9	30.4	40.3	32.5	44.7	22.7	34.6	37.7	46.2	43.1	49.7	27.3	39.4	43.6	50.0	44.9

[1] Sample seperated into 2 parts which could not be refluxed.

[2] Sample broke, not a normal test.

All of these extremely hard residues had little if any ductility. All of the final residues had high percentages of material insoluble in 86° B. naphtha and had, especially in the case of the cracked products, including sample 6, large amounts of material insoluble in carbon disulphide and carbon tetrachloride. The percentage of insoluble material was greatest in the residues of the thinnest films, indicating that greater internal alterations had taken place in these samples.

The percentages of material insoluble in 86° B. naphtha in the residues after exposure are plotted in figure 7. As the volatile constituents of the liquid asphalts evaporated, the material insoluble in naphtha was concentrated in the residues. Assuming no change except that resulting from loss of volatile matter, the theoretical percentages of material insoluble in naphtha that should be present in the different residues are shown in figure 7. Only in the case of samples 1 and 5 was there an appreciable theoretical increase in insoluble material caused solely by loss of volatile matter and the resulting concentration.

The curves show that all of the residues from exposure contained considerably more than the calcuated amount of material insoluble in 86° B. naphtha. The percentage of material insoluble in naphtha actually present in the residues of the 1/32-inch films after 15 weeks was twice the calculated percentage for sample 1 and 8.7 times the calculated percentage for sample 6.

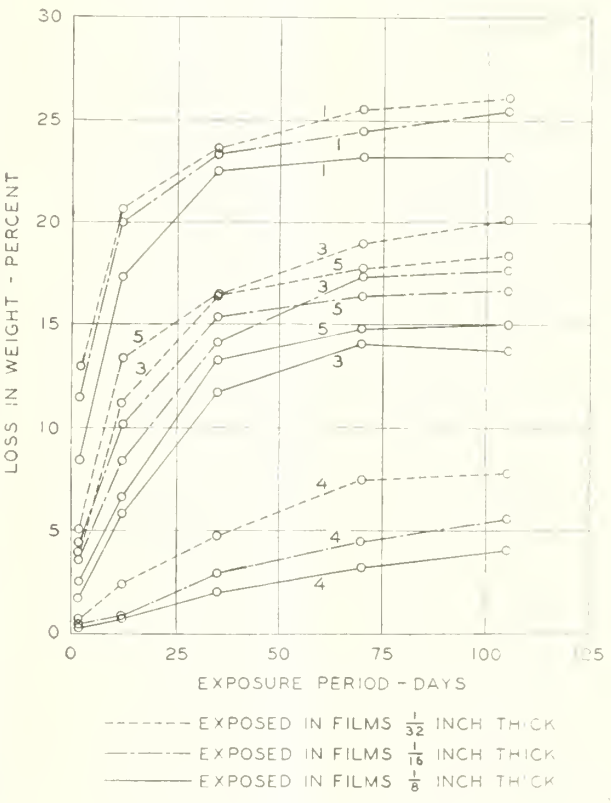

FIGURE 6.—RATE OF LOSS BY ASPHALTIC MATERIALS EXPOSED IN THIN FILMS.

MODIFIED OLIENSIS SPOT TEST USED IN STUDYING EXPOSURE RESIDUES

One sample of each material was also exposed in a ⅛-inch film in the open air adjacent to the circular track. These samples were tested at the end of 15 weeks and the results are given in table 5. Sample 6 became quite granular in texture and greasy in appearance, but the incompatibility of the blend was not as pronounced as in the exposure-box tests. All of the materials had considerably less loss in weight than occurred in the distillation test and developed residues much harder than those from distillation. Sample 1 lost the most and became the hardest. While samples 3 and 5 lost about equal amounts, sample 5 evidenced the greater hardening properties.

That the asphaltic residue test cannot be used to estimate the amount of 100 penetration asphalt a liquid asphalt will ultimately develop under certain conditions is illustrated by the behavior of sample 1. In the

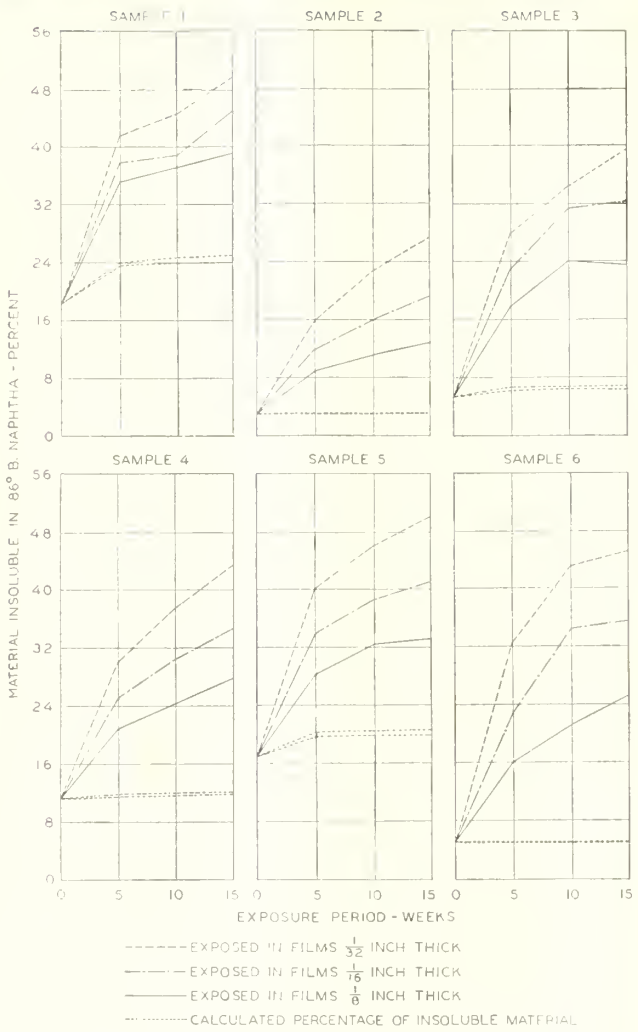

FIGURE 7.—PERCENTAGES OF MATERIAL INSOLUBLE IN 86° B. NAPHTHA IN THE RESIDUES AFTER VARIOUS PERIODS OF EXPOSURE.

asphaltic residue test this sample lost 32.9 percent and in the open-air exposure the loss was only 19.7 percent, yet the two residues had comparable consistencies as measured by the penetration test. The results indicate that 15 weeks of exposure in the open air was slightly less severe than 5 weeks of exposure in the glass-covered boxes.

In a previous study of liquid asphalts, it was found that all of the materials that were originally homogeneous became heterogeneous after 5 weeks of exposure under conditions similar to those existing in the exposure tests in glass-covered boxes described in this report. The heterogeneity apparently increased, but the spot test made in accordance with the original description of the Oliensis method [4] permitted only a rough approximation to be made of the degree of heterogeneity and rate of increase in the residues exposed.

At the 1936 meeting of the American Society for Testing Materials, Oliensis [5] described a modification of his method that permits the degree of heterogeneity to be determined quantitatively. When an asphaltic material gives a positive stain after dispersion in the standard 50° B. naphtha, the material may be considered as heterogeneous. If xylene is added to the naphtha in increasing amounts and the asphaltic material is dispersed in the same volume of the blended solvent as of the naphtha, a concentration may be reached that will produce a negative stain. The degree of heterogeneity may then be expressed by the percentage of xylene that must be added to the standard naphtha in order to produce a negative stain. This percentage is called the xylene equivalent.

In practice the laborious task of determining the exact percentage of xylene at which the change from positive to negative occurs is not justified, and, therefore, the xylene equivalent is generallly expressed by limiting percentages. Thus, a xylene equivalent of 50–55 means that with 50 percent xylene the material gave a positive stain, and with 55 percent xylene a negative stain was obtained. If a positive spot is obtained with xylene alone, the degree of heterogeneity is not determinable with this solvent.

TABLE 5.—*Results of tests on ⅛-inch films of liquid asphalt exposed in open for 15 weeks*

Sample no.	1	2	3	4	5	6 [1]
Loss in 15 weeks—percent	19.7	−2.5	9.1	0.1	10.6	1.0
Float at 122° F.—seconds	1,312	50	75	50	343	54
Penetration at 77° F., 100 grams, 5 seconds	108	-----	-----	-----	+300	-----
Softening point—° F.	123	-----	-----	-----	124	-----
Ductility at 77° F.—centimenters	25	-----	-----	-----	-----	-----
Organic matter insoluble in CS_2—percent	0.48	0.21	0.17	0.77	0.89	0.88
Organic matter insoluble in CCl_4—percent	0.47	0.17	0.14	2.23	3.92	1.75
Organic matter insoluble in 86° B. naphtha—percent	33.0	8.1	16.1	19.8	27.4	16.2

[1] Material granular and nonuniform in texture.

XYLENE EQUIVALENT INCREASED AS EXPOSURE TIME INCREASED AND AS FILM THICKNESS DECREASED

As previously stated, samples 1, 2, and 3 originally and after distillation gave negative stains when dispersed in 50° B. naphtha. Samples 4, 5, and 6 gave positive stains not only with the standard solvent but also with 100 percent xylene. These three materials were, therefore, outside the scope of the quantitative test for heterogeneity.

Samples 1, 2, and 3, when exposed under glass for 5, 10, and 15 weeks and in the open air for 15 weeks, produced residues which gave positive stains with the standard naphtha. The xylene equivalents of the residues from the exposures under glass are given in table 6.

It is seen that the xylene equivalent at the end of each test period increased as the film thickness decreased. For each film thickness, except the ⅛- and 1/16-inch films of sample 1, the xylene equivalent increased as the time of exposure increased. The reason for the slightly lower xylene equivalents obtained on the 10-week residues of the ⅛- and 1/16-inch films of sample 1, as compared to those obtained on the 5-week residues of the same film thickness, cannot be explained.

If the solubilities of the residues that produced xylene-insoluble spots are compared with the solubilities of the 1/32-inch film of sample 2 after 15 weeks of exposure (table 4), it is seen that this residue of sample 2 had a larger percentage of material insoluble in carbon disulphide than two of the residues reacting positive in 100 percent xylene and more material insoluble in carbon tetrachloride than three of the residues whose heterogeneity was not determinable by this method.

[4] A Qualitative Test for Determining the Degree of Heterogeneity of Asphalts, by G. L. Oliensis. Proceedings of the American Society for Testing Materials, 1933.

[5] A Further Study of the Heterogeneity of Asphalts—A Quantitative Method. Proceedings of the American Society for Testing Materials, 1936. *Also see* The Oliensis Spot Test Improved, by R. H. Lewis and J. Y. Welborn. PUBLIC ROADS, vol. 18, no. 3, May 1937.

TABLE 6.—*Xylene equivalents of residues of homogeneous materials exposed under glass*

Sample no.	Film thickness	Xylene equivalents after exposure for—		
		5 weeks	10 weeks	15 weeks
	Inch			
1	⅛	4–8	0–2	8–16
	1/16	8–12	4–8	(1)
	1/32	12–16	(1)	(1)
2	⅛	0–2	16–20	36–40
	1/16	28–32	36–40	56–60
	1/32	48–52	60–64	68–72
3	⅛	24–28	32–36	36–40
	1/16	36–40	44–48	48–52
	1/32	48–52	64–68	(1)

[1] Xylene insoluble.

The residue of sample 2, however, had a xylene equivalent of 68–72. Evidently there are essential differences in the character of the carbenes and free carbon in these residues that have a direct bearing on their ready dispersion in either 100 percent xylene or the naphtha-xylene blend.

The SC–2 type of liquid asphaltic material, when used in the construction of oil-processed roads, is generally mixed with aggregate graded in size from 1 inch maximum to dust. Fine aggregate mixtures, however, were used in this investigation because greater uniformity in the composition of the mixed surfaces could be obtained in the relatively small test sections, and the fine-graded aggregate is more suitable for the preparation of both Hubbard-Field stability and toughness specimens.

Preliminary tests were made on an indoor circular track with mixtures containing various proportions of Potomac River sand, limestone dust, and the California residual, sample 3. A mixture containing 85 percent of sand and 15 percent of dust, by weight, was used. This aggregate, when separated on standard sieves, had the following grading:

	Percent
Passing no. 10, retained on no. 20	10.2
Passing no. 20, retained on no. 30	14.1
Passing no. 30, retained on no. 40	20.9
Passing no. 40, retained on no. 50	20.4
Passing no. 50, retained on no. 80	15.0
Passing no. 80, retained on no. 100	1.8
Passing no. 100, retained on no. 200	2.6
Passing no. 200	15.0

This aggregate, when combined with 7.5 percent of oil, produced a mixture having sufficient stability to withstand considerable traffic without undue displacement. This percentage by weight of asphaltic material was equal to 18.3 percent by volume and this percentage by volume was used in all mixtures. Because of the differences in the specific gravities of the bituminous materials, the percentage of oil by weight of the mixture varied from 7.3 for sample 2 to 8.1 for sample 5. In this way the void-filling capacity and film thickness of the asphaltic materials were held constant for all of the mixtures.

Sufficient amounts of each mixture were made to construct the track sections and provide excess material for Hubbard-Field stability and toughness cylinders. Additional cylinders 2½ inches high and 2 inches in diameter were also prepared and stored in forms, their upper surfaces being exposed in the immediate vicinity of the track to the same climatic conditions as the track sections. These cylinders were used to replace cores taken from the track surfaces during the course of the experiment.

TABLE 7.—*Results of tests on Hubbard-Field cylinders*

Age when tested (weeks)	Loss of bitumen by sample no.—						Stability at 77° F. of sample no.—					
	1	2	3	4	5	6	1	2	3	4	5	6
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*
Immediately							375	375	325	400	425	350
AGED IN OVEN AT 140° F.												
1	12	3	7	3	7	3	1,050	425	525	525	775	425
2	14	3	7	2	9	3	1,425	350	550	500	900	500
3	16	3	8	3	10	3	1,850	525	750	625	1,025	525
4	17	3	9	4	11	4	1,825	475	775	625	1,150	525
5	18	3	10	4	12	4	2,200	525	875	750	1,250	550
AGED IN GLASS-COVERED EXPOSURE BOXES												
1	8	1	3	1	4	1	825	425	500	450	850	475
3	15	1	8	2	9	2	1,900	500	900	750	1,525	700
5	18	0	9	2	10	1	2,700	575	1,200	875	1,850	725
10	20	1	11	3	12	3	3,500	625	1,600	1,200	2,150	1,050
15	22	1	13	4	13	3	4,100	800	2,200	1,725	2,650	1,325
20	21	0	12	4	13	2	4,175	750	2,000	1,600	2,600	1,100
AGED IN CANVAS-COVERED EXPOSURE BOXES												
1	3	0	1	0	0	0	500	475	350	450	650	425
3	6	0	2	1	3	0	725	600	450	450	775	450
5	7	0	1	0	3	0	800	475	425	425	775	450
10	11	0	2	0	5	1	1,200	575	500	425	900	500
15	13	0	4	0	6	1	1,550	600	675	500	1,100	575
20	12	0	3	0	5	0	1,525	450	600	525	1,100	575
AGED IN OPEN AIR ADJACENT TO TRACK												
1	4	0	1	0	2	0	525	450	400	475	650	400
3	8	1	3	1	4	1	850	625	575	425	850	475
5	10	1	4	0	5	0	1,250	550	600	550	1,000	475
10	14	1	4	1	7	1	1,550	650	800	575	1,225	500
15	16	1	7	1	8	1	2,050	750	1,100	725	1,550	675
20	15	0	6	1	7	1	1,900	650	1,000	675	1,425	675
AGED IN OPEN AIR, ALTERNATELY WET AND DRY												
20							1,925	550	600	700	1,300	750

CYLINDERS AGED IN GLASS-COVERED EXPOSURE BOXES HAD HIGHEST STABILITIES

The stability cylinders were made in sets of three. Each cylinder, containing about 100 grams of aggregate, was compacted at 77° F. under a load of 3,000 pounds per square inch. One set of cylinders of each sample was immediately tested for stability at 77° F. Five sets were aged in an oven at 140° F., and one set of each was tested for stability at 77° F. at the end of 1, 2, 3, 4, and 5 weeks. Six sets of cylinders of each mixture were aged in glass-covered exposure boxes, six sets in canvas-covered exposure boxes, and six sets in the open air adjacent to the track. One set from each condition of exposure was tested at the end of 1, 3, 5, 10, 15, and 20 weeks.

All samples were weighed before and after aging and the loss in weight was expressed as the percentage of bitumen lost. One set of cylinders for each mixture was also aged out of doors near the track for 20 weeks in a large shallow pan. The cylinders were covered with water every night and exposed uncovered during the day after the removal of the water each morning. At the end of the test period the condition of the specimens was noted and each was tested for stability.

The results of the Hubbard-Field stability test given in table 7 indicate that, although the cylinders of all the samples had about the same initial stability, the final

TABLE 8.—*Results of tests on bitumen recovered from Hubbard-Field cylinders after aging*

Sample no.	1	2	3	4	5	6
Original asphaltic material dissolved in benzol and recovered:						
Float at 77° F.—seconds	29	187	35	55	48	63
Organic matter insoluble in CCl_4—percent	0.06	0.04	0.05	0.20	2.06	0.16
Organic matter insol. in 86° B. naphtha—percent	19.1	3.7	5.4	12.0	18.1	7.4
Bitumen recovered from cylinders aged for 20 weeks in glass-covered boxes:						
Float at 122° F.—seconds	1,084	45	100	63	1,200	203
Penetration at 77° F., 100 grams, 5 seconds	122				245	
Organic matter insoluble in CCl_4—percent	0.24	0.41	0.14	1.82	0.46	0.85
Organic matter insoluble in 86° B., naphtha—percent	30.5	7.6	18.6	22.4	25.8	14.9
Bitumen recovered from cylinders aged for 20 weeks alternately wet and dry:						
Float at 77° F.—seconds	683	692	75	114	395	2,400+
Float at 122° F.—seconds	100	35	30	31	56	57
Organic matter insoluble in CCl_4—percent	0.20	0.07	0.14	0.40	0.22	0.93
Organic matter insoluble in 86° B., naphtha—percent	24.1	5.0	9.8	16.8	21.8	13.1

stabilities under any method of aging varied greatly. Cylinders of sample 1 had the highest ultimate stability and percentage of loss of bitumen, and those of sample 5 had the next highest stability and percentage of loss. Cylinders of sample 3, although losing almost as much as the cylinders of sample 5, had only slightly higher stability than the cylinders of sample 4, which had a small loss of volatile matter under all methods of aging. Cylinders of samples 2 and 6 lost little and developed comparatively little stability. Cylinders of sample 6 had slightly greater stabilities than those of sample 2.

The final stabilities under each method of aging, plotted in figure 8, show that aging in the glass-covered exposure boxes gave the highest stability, and aging in the canvas-covered boxes generally gave the lowest stability. The stabilities obtained by aging in the oven at 140° F. for 5 weeks, by aging in the open air for 20 weeks, and by aging under alternately wet and dry conditions for 20 weeks, were all about the same except in the case of sample 3. The stabilities of the specimens of sample 3, exposed alternately wet and dry, were as low as those of the specimens exposed under canvas covers.

The specimens exposed under alternately wet and dry conditions varied somewhat in behavior. The cylinders of sample 1 had some slight cracks and had swelled so that it was difficult to place them in the stability testing mold. Those of sample 2 had also swelled and half of the specimens had split in two horizontally. The cylinders of samples 3 and 4 had small blisters on top while those of samples 5 and 6 remained in good condition. Although the outer surfaces of the specimens of the other five samples were dry and dull colored, those of sample 6 were always glossy-black and greasy, probably because of the tendency of this blend to separate into two parts and the subsequent movement of the oily fraction to the surface. This same behavior was also noted in the stability specimens exposed adjacent to the track.

The behavior of the cylinders under these various conditions indicated that the hardening properties of sample 1, and to a lesser extent samples 3, 4, and 5, might vary considerably under various temperature and climatic conditions. Samples 2 and 6, because of their low volatility in the laboratory heat tests, would not be expected to show a great difference in behavior under such conditions as might normally occur in actual use.

After the specimens that had been exposed for 20 weeks in the glass-covered boxes and those exposed under alternately wet and dry conditions had been tested for stability, their bitumen was extracted with benzol, recovered by Abson's method [6], and tested for consistency and solubility. Some of the original material was dissolved in benzol and recovered in the same manner for comparative purposes. These results are given in table 8. The high temperatures existing at times in the glass-covered exposure boxes developed residues in the cylinders which, when extracted, were much harder than the distillation residues of the original liquid asphalts and, in the case of samples 1 and 5, were semisolid.

The bitumen recovered from the cylinders exposed under alternately wet and dry conditions was much harder than the original material although, except for samples 4 and 6, it was softer than the distillation residue. Comparisons of the percentages of material insoluble in 86° B. naphtha in the bitumen from the cylinders after alternately wet and dry exposure and the percentages of insoluble matter in the distillation residues indicate that in some cases considerable alterations occurred in the structure of the bitumen exposed under these conditions.

RESULTS OF TESTS ON TOUGHNESS CYLINDERS INDICATED VARIABLE CHARACTERISTICS OF THE MATERIALS

The Page impact test was used to obtain further information on the behavior of these mixtures after exposure. This test has been used in previous studies of bituminous mixtures in the Bureau laboratory, and in a report by Reeve and Lewis [7], it was indicated that this test offered possibilities as a means for determining in advance the relative service behavior of bituminous concretes.

The impact cylinders used in this investigation consisted of six sets of three specimens each for each mixture. The cylinders, 1 inch in diameter and 1 inch high, were prepared in the same manner as the Hubbard-Field stability cylinders. They were aged in the glass-covered boxes and one set was tested at the end of 1, 3, 5, 10, 15, and 20 weeks of exposure. The test was made at 77° F. using the Page impact machine equipped with a 500-gram hammer and a 110-gram plunger instead of the 500-gram hammer and 1-kilogram plunger used in the work of Reeve and Lewis [7] and the 2-kilogram hammer and 1-kilogram plunger normally used in testing rock. Even with these smaller weights some of the specimens could not support the plunger and the impact value was reported as zero.

The results of these tests, given in table 9, show that cylinders containing samples 2 and 6 developed little resistance to impact. Cylinders containing samples 1 and 3 developed maximum toughnesses of 13 and 9, respectively. The cylinders containing samples 4 and 5 first showed an increase in toughness, and then, as the time of exposure increased, lost in toughness. After 20 weeks of exposure under glass the cylinders of sample 1 had the highest stability and the highest toughness.

The cylinders of sample 5, although having three and one-half times greater stability than cylinders of sample 2, had the same resistance to impact. Although the stabilities of the cylinders of samples 4 and 5 showed no great change as the time of exposure increased, the gradual loss in toughness by the cylinders of sample 5 and the rather abrupt loss in resistance to impact by

[6] Method and Apparatus for Recovery of Asphalt. 1933 Proceedings American Society for Testing Materials.

[7] Toughness of Bituminous Aggregates, Journal of Agriculture Research, vol. X, no. 7, August 1917.

TABLE 9.—*Results of impact tests on cylinders aged under glass*

Age when tested (weeks)	Loss of bitumen by sample no.—						Height of drop at failure for sample no.—					
	1	2	3	4	5	6	1	2	3	4	5	6
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Cm*	*Cm*	*Cm*	*Cm*	*Cm*	*Cm*
1	11	2	6	0	7	0	0	0	0	0	0	0
3	21	1	9	1	11	0	12	0	3	3	5	0
5	22	1	11	6	13	5	12	1	7	5	4	1
10	22	0	13	-----	12	-----	13	2	5	10	4	2
15	23	1	16	5	14	3	13	2	9	4	3	1
20	22	0	15	4	14	3	12	2	8	5	2	1

cylinders of sample 4 undoubtedly result from changes occurring in the residual binders.

The Bureau's circular track, shown in figure 9, was used for testing the materials under weathering and traffic. This track was briefly described in the January 1934 issue of PUBLIC ROADS. It consists essentially of an annular concrete trough 12 inches deep, 18 inches wide, and 12 feet in diameter at the center line. Two full-size automobile wheels, fixed to the two ends of a rigid structural member rotated in a horizontal plane by a vertical shaft at the center of the track, provided the traffic. The load on each tire was 800 pounds. A hand-wheel adjustment made it possible to shift the path of the wheels to any position on the track surface. For compaction a speed of 4½ miles per hour was used, and for all subsequent applications of traffic a speed of 6 miles per hour was used.

The track was divided into 6 sections numbered 1 to 6, corresponding to the samples of asphaltic material. The surfaces were placed June 6, 1934, upon a sand-choked, trap-rock base on which a thin layer of sand-cement mortar had been placed to provide a smooth, somewhat porous base. The surfaces were compacted the following day by means of 1,000 wheel-trips of traffic distributed over the surface. At this time the surfaces of sections 1, 3, and 5 were well closed, smooth, and uniform in appearance. The surface of section 4, although slightly drier and more open, was also smooth and uniform in appearance. The surfaces of sections 2 and 6 were smooth but somewhat nonuniform in color and had a greasy, noncohesive appearance.

MIXTURES ON CIRCULAR TRACK REMAINED IN GOOD CONDITION DURING EXPOSURE TO WEATHER AND TRAFFIC

The sections were subjected to traffic only 1 day each week, when 2,000 wheel passages of traffic were applied, distributed over the surface. The track was covered at night and during inclement weather by a portable shed. Traffic was applied weekly until October 1, 1934, when testing was suspended for the winter. During this period the maximum daily temperature in the surface mixtures ranged from 82° to 144° F. with an average daily maximum of 114° F.

In the very early stages of the test, section 1 developed quite a tough, well-bonded crust which gradually increased in hardness. Surface cracking developed in this section during the sixth week after construction. On hot days some subsurface movement of the mixture could be detected under traffic. The condition of section 1, except for the cracks that developed, remained good throughout the test.

The surface of section 2 was picked up by the tires during the first weekly application of traffic. This condition necessitated the use of canvas covers during the traffic runs up to and including the ninth week, after which time the surface did not pick up under traffic. Slight cracking developed in the sixth week. Some raveling was noticeable until the twelfth week when virtually all raveling ceased, the surface remaining well cemented thereafter.

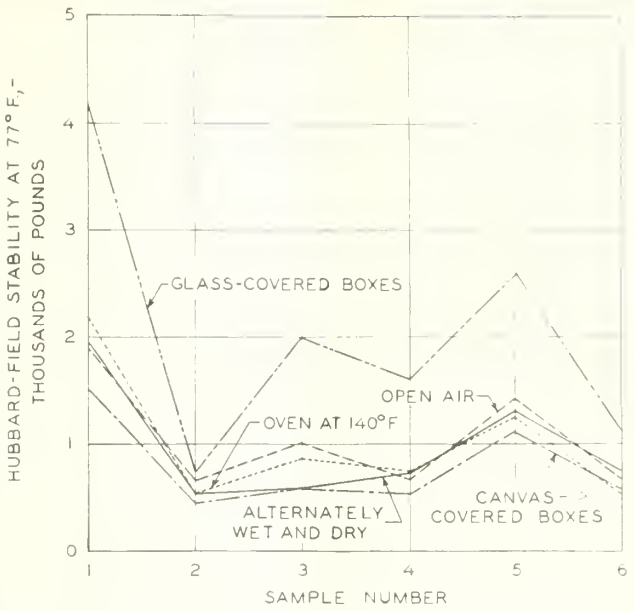

FIGURE 8.—FINAL STABILITIES OF HUBBARD-FIELD CYLINDERS AFTER VARIOUS METHODS OF AGING. ALL CYLINDERS TESTED AFTER AGING FOR 20 WEEKS EXCEPT THOSE AGED IN OVEN WHICH WERE TESTED AFTER 5 WEEKS.

Section 3 raveled slightly during the first weekly application of traffic. After that no further raveling was noted except for one small area which raveled during the seventh weekly traffic run and healed during the eighth traffic run. Hair cracks developed during the fourth week. Considerable subsurface movement under traffic was noted from the twelfth week to the end of the summer but the general appearance of the section continued to be good.

Section 4 remained in good condition throughout the summer. Hair cracks developed during the fifth week and were noticeable until the twelfth week when they became cemented and did not again develop. Section 5 also maintained a good appearance although the cracks that developed during the fourth week remained unhealed to the end of the summer.

Section 6 developed a surface similar to section 2 and canvas covers were used on it during traffic up to and including the ninth weekly traffic run. The surface was then slightly greasy and still raveled to some extent. This raveling continued with slight moderation until the end of the summer.

As the appearance of the sections at the end of the summer of 1934 did not indicate that there had been any failures caused by weathering of the asphaltic binders, the surfaces were again exposed beginning on May 27, 1935, and again subjected to 2,000 wheel passages weekly until September 27, 1935. During this period the maximum daily air temperature varied from 64° to 98° F. with an average daily maximum of 86° F.; the maximum daily temperature of the mixtures one-half inch below the surface varied from 65° to 151° F. with an average daily maximum of 126° F.; and at 1½ inches below the surface the maximum daily temperature varied from 64° to 142° F. with an average daily maximum of 117° F.

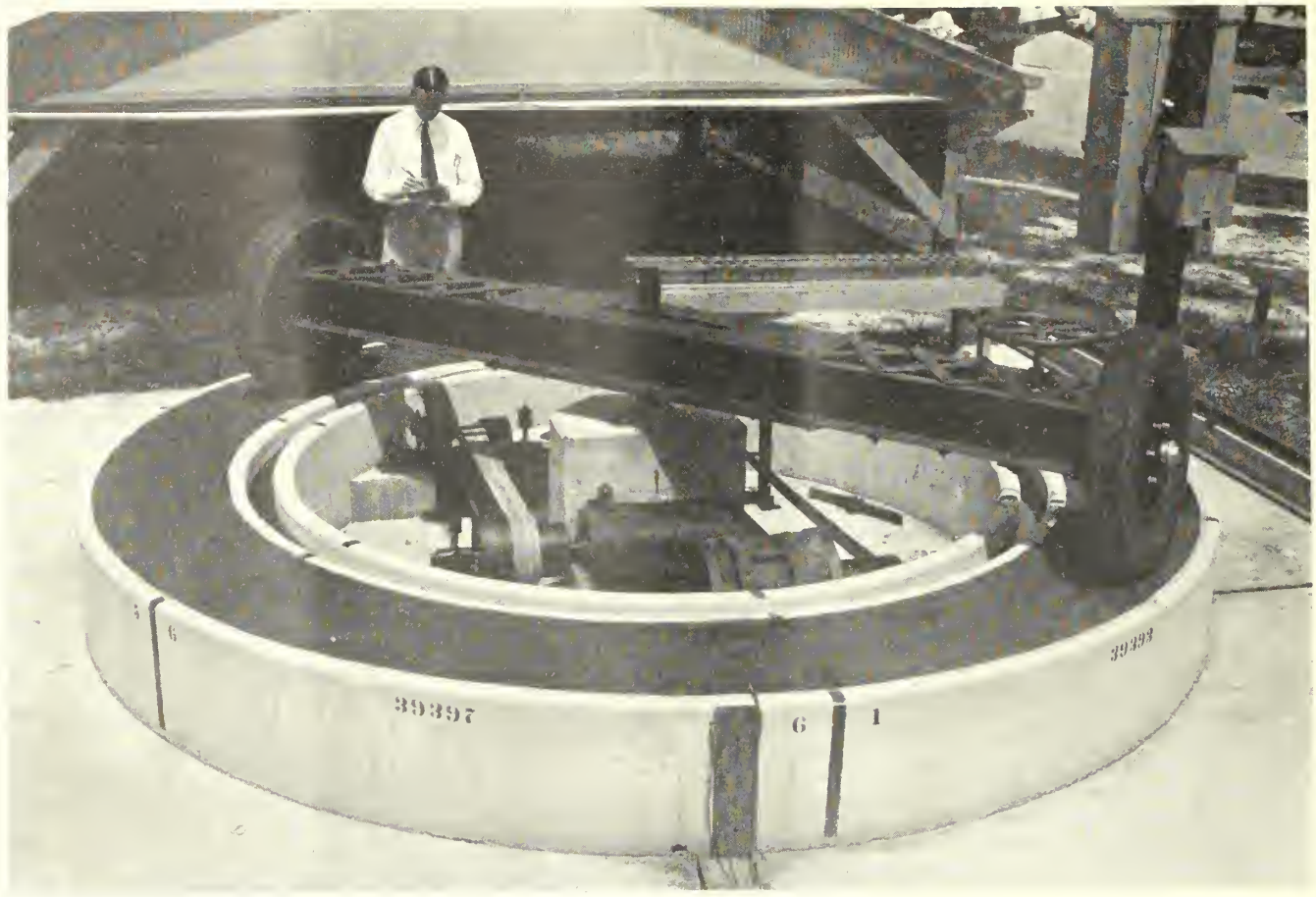

FIGURE 9.—OUTDOOR CIRCULAR TRACK USED IN THE WEATHERING TESTS.

Since the surfaces had become fairly well bonded, the shed that had been used to protect the track at night and on rainy days was removed on the first of July for the balance of the test. No noticeable changes occurred in the appearance of the surfaces from week to week and on September 27, 1935, after 70,000 wheel-passages, the appearance and condition of the surfaces were good, as shown in figure 10.

The track sections shown in figure 10 are briefly described as follows:

Section 1: Gray, hard, and well-bonded, with open alligator cracks; soft and yielding underneath.

Section 2: Black and pliable with a thin, easily broken surface skin having very indistinct cracks; soft and yielding underneath; asphalt seeped through the hard cracks slightly under traffic.

Section 3: Grayish black, well-bonded and fairly tough with slight alligator cracks; soft and yielding underneath.

Section 4: Grayish black, firm and well-bonded with almost indistinguishable hair cracks; firm underneath.

Section 5: Grayish black, firm and well-bonded with indistinct hair cracks; a black hairline of asphalt along each crack showed a tendency to heal by seepage and hardening of the asphalt.

Section 6: Black and greasy; firm with no cracks but not well cemented; surface raveled if broken slightly; firm underneath.

TRAFFIC ON ALTERNATELY WET AND DRY TRACK CAUSED FAILURE OF CERTAIN MIXTURES

As the asphaltic materials had shown considerable variations in characteristics and behavior, as measured by the results of other tests and the track sections containing them had shown no great variations in behavior under the traffic test, it was decided to change the character of the traffic test. After making permeability tests, which indicated that all of the surfaces were impervious to water, traffic was applied in increments of 6,000 wheel passages daily. The surfaces of the sections were sprinkled with water 1 day during the period of traffic application; the following day traffic was applied with the surface dry. Thus the track was alternately wet 1 day and dry the next during the application of traffic.

No changes were noted in the condition of the sections until the third application of traffic on the wet track when sections 2 and 6 began to ravel, section 2 raveling the most. This raveling diminished considerably the following day when the track was dry. The asphaltic material seemed to have been stripped from the sand particles on the surface. Section 3 started to ravel on the fourth application of traffic on the wet track, although the sand displaced from the surface of section 3 appeared to carry more oil than that thrown off the surfaces of sections 2 and 6. Sections 1, 2, 3, and 6, under continued traffic, tended to shove and push over the curb. This behavior was least pronounced on section 1. After the fourth application of traffic on the alternately wet and dry track, sections 2 and 6 had both raveled so extensively that the test was discontinued although sections 4 and 5 had still shown no tendency to displace or ravel. Tests showed that all of the sections were still impermeable to water. The appearance of the surface of each section is shown in figure 11.

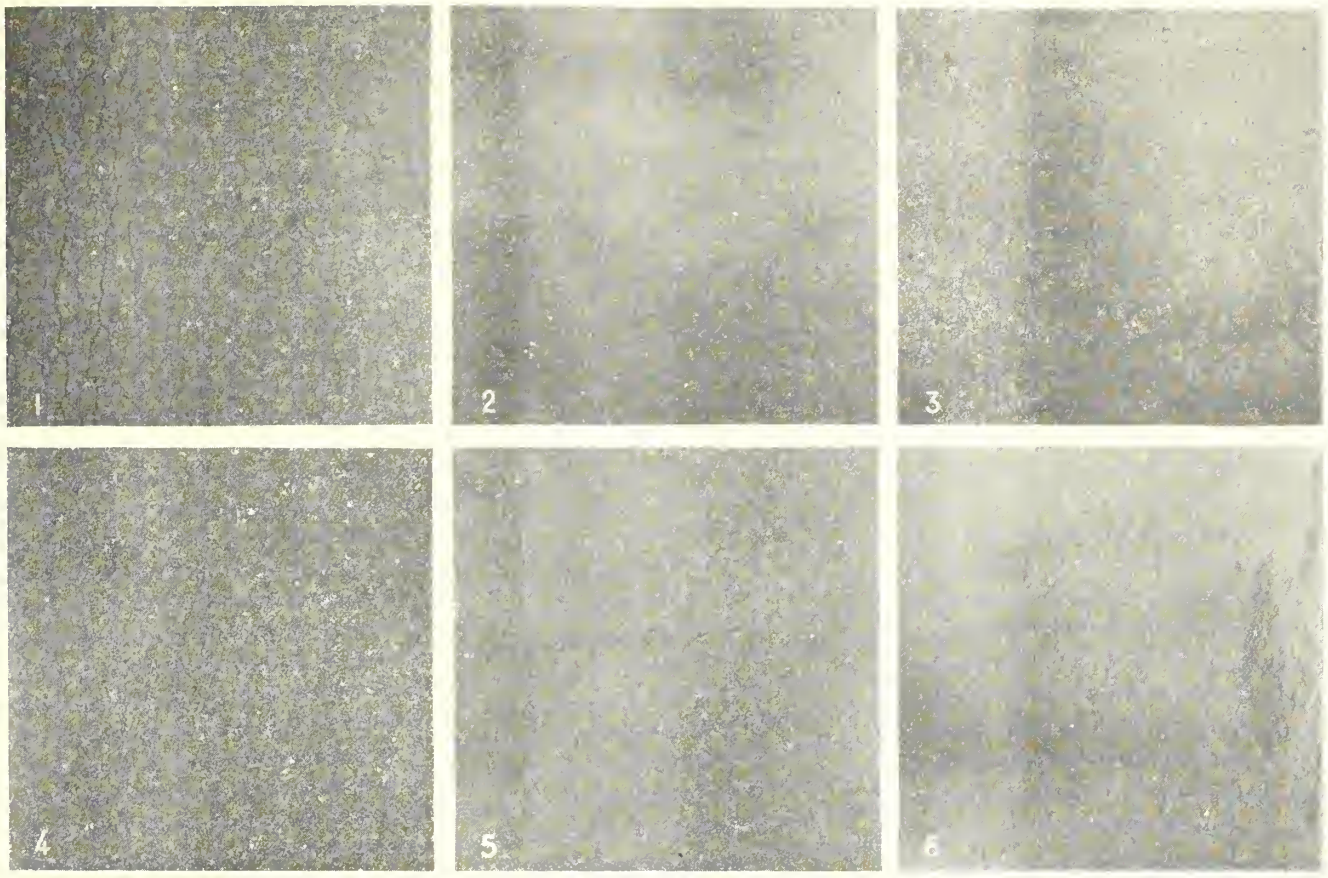

FIGURE 10.—APPEARANCES OF THE TRACK SECTIONS AFTER THE APPLICATION OF 70,000 WHEEL-PASSAGES OF TRAFFIC.

From time to time 2-inch cylinders the full depth of the surfaces were cored from the track sections and the upper inch was tested for stability at 77° F. in the Hubbard-Field machine. The holes from which the cores were taken were refilled with the previously molded cylinders. The results of these stability tests are given in table 10.

After the completion of the final traffic tests, a portion of each section was removed for extraction of the soluble asphaltic material, in the same manner as previously described, and the recovered bitumen analyzed. The results of these tests are given in table 11. Layers of the surface one-half inch thick were used for analysis. Section 1 was the only section that had an appreciable surface crust. This crust was approximately one-eighth inch thick and the bitumen therein was recovered separately, in addition to the bitumen recovered from the top ½-inch layer which included this crust. Sections 1, 2, and 3 possessed varying degrees of plasticity throughout their entire thickness, section 1 being the most plastic, followed by section 3 and then section 2, which, although greasy, was slightly plastic. Sections 4, 5, and 6 were friable and crumbly, section 6 having a very dry, somewhat greasy appearance.

Comparison of tables 7 and 10 shows that only the cores of sections 2, 4, and 6 had greater stabilities at the conclusion of the test than did the molded cylinders containing the same asphaltic binder and exposed under glass for 20 weeks. Stability is developed by an increase in the density of the mixture or by an increase in the hardness of the contained bitumen. The molded cylinders increased in stability only through an increase in the hardness of the bitumen by loss of volatile matter and other alterations. This increase in hardness, as previously shown, was considerable, especially by the cylinders of samples 1 and 5.

TABLE 10.—*Results of Hubbard-Field stability test at 77° F. on cores taken from the track sections*

Date cored	Sample no.					
	1	2	3	4	5	6
	Lb.	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*
June 8, 1934 (immediately after compaction)	350	400	200	250	225	225
June 12, 1934 (after 1 week)	300	250	150	250	350	250
June 26, 1934 (after 3 weeks)	500	300	250	300	350	250
July 10, 1934 (after 5 weeks)	600	425	400	500	575	450
Aug. 14, 1934 (after 10 weeks)	1,125	875	650	750	950	700
Sept. 19, 1934 (after 15 weeks)	1,525	1,250	700	1,100	950	950
May 27, 1935 (before 1935 exposure)	1,950	1,250	975	1,250	1,075	1,125
Sept. 27, 1935 (before traffic on alternately wet and dry track)	2,300	1,675	1,500	1,700	1,425	1,275
Dec. 9, 1935 (final tests)	2,375	1,800	1,450	1,825	1,550	1,650

The track surfaces, on the other hand, increased in stability both from an increase in hardness of the bitumen and from increased density caused by traffic. The bitumen extracted from the surfaces was in no case as hard as the bitumen extracted from the molded cylinders exposed under glass. In no case, except the top ¼-inch layer of section 1, were the extracted bitumens appreciably harder than the residues from distillation of the original materials. In the case of the sections containing samples 2, 3, 4, and 6 the extracted bitumen was of approximately the same consistency as the distillation residue. In the case of the section containing sample 1 (exclusive of the ¼-inch crust), and of sample 5, the extracted bitumen was softer than the distillation residue.

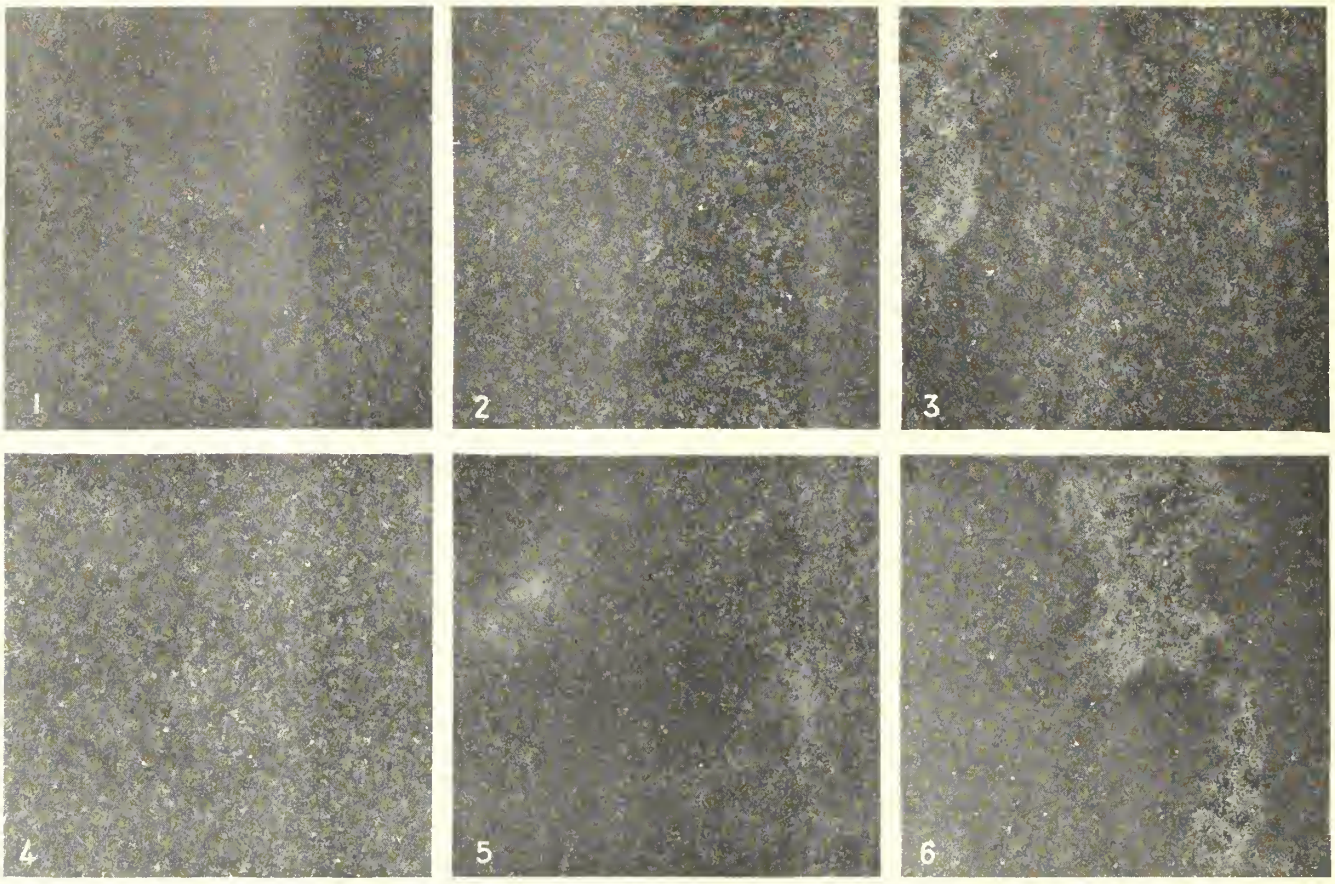

FIGURE 11.—APPEARANCES OF TRACK SECTIONS AT END OF TEST.

TABLE 11.—*Results of tests on bitumen recovered from circular track sections after completion of traffic tests*

Section	Asphaltic material in mixture when laid [1]	Layer	Bitumen extracted [1]	Tests on recovered bitumen			
				Float at 122° F.	Organic matter insoluble in CS_2	Organic matter insoluble in CCl_4	Organic matter insoluble in 86° B. naphtha
	Percent		*Percent*	*Seconds*	*Percent*	*Percent*	*Percent*
1	7.5	Top ⅛ inch	7.1	232	0.04	0.08	31.5
		Top ½ inch	7.1	80	.04	.03	25.7
		Second ½ inch	7.0	50	.06	.04	25.1
		Third ½ inch	6.7	39	.07	.03	24.6
2	7.3	Top ½ inch	7.7	36	.04	.08	8.5
		Second ½ inch	7.4	36	.08	.11	7.3
		Third ½ inch	6.9	43	.12	.10	7.6
3	7.5	Top ½ inch	7.3	38	.08	.04	14.3
		Second ½ inch	7.2	26	.21	.21	11.9
		Third ½ inch	7.2	26	.06	.04	11.1
4	8.1	Top ½ inch	7.4	39	.09	1.07	18.0
		Second ½ inch	7.3	32	.14	1.01	16.9
		Third ½ inch	8.1	25	.08	.76	16.6
5	7.8	Top ½ inch	7.5	56	.08	.75	21.2
		Second ½ inch	7.5	49	.11	.88	20.5
		Third ½ inch	7.1	66	.12	.82	21.5
6	7.7	Top ½ inch	7.0	38	.27	.52	10.4
		Second ½ inch	6.9	33	.17	.25	9.6
		Third ½ inch	6.5	35	.20	.99	10.4

[1] Percentage by weight.

The high stabilities of the final cores from the track sections containing samples 2, 4, and 6, when compared with the low stabilities obtained on the cylinders after 20 weeks of exposure under glass (in which the asphaltic materials had hardened to a greater extent), indicate that the higher stabilities of the cores from the circular track sections must result chiefly from the increased densities that developed under the action of traffic. Although a hard outer crust was formed on the molded specimens, no appreciable crust was formed on any of the track sections except section 1. This is shown for sections 2, 3, and 6 by the fact that, although raveling removed the surfaces to a great extent upon sections 2 and 6 and to a lesser extent on section 3 during the final application of traffic on the alternately wet and dry track, results of stability tests on the final cores taken from the track sections showed that no change in stability occurred during this period.

SUMMARY

While the purpose of this investigation was to correlate the service behavior of these selected slow-curing, liquid asphalts and road mixtures containing them with the results of the usual laboratory tests and with the results of the exposure tests, there are certain factors that must be considered in interpreting the test data already presented.

It is believed that the conditions of the circular track weathering test were not as severe as those to which mixtures of similar character would be subjected if placed as actual road surfaces for the following reasons:

1. The surfaces received lateral support from the curbs.
2. The sections were laid on a dry, stable, nonflexible base.
3. The traffic applied subjected the surfaces to practically no impact or mutilative forces.
4. Comparatively light wheel loads were applied.

The fine-graded mixtures compacted to produce a dense, tight surface so that loss of volatile matter and weathering of the asphaltic binders were greatly retarded.

The conditions of the exposure tests, especially those made on the thin films and molded specimens in the glass-covered boxes, were more severe than those that occur in the circular track test or under actual road conditions. Accordingly, the extreme differences in ultimate behavior of the track sections, which the results of the exposure tests on both thin films and molded specimens indicated might occur, did not develop within the period these sections were under test.

The great potential differences in hardening properties of the materials investigated are shown by the results of the laboratory tests as well as by the exposure test results. It is clearly indicated that the distillation requirements for SC–2 material and the consistency requirements for the residue from distillation are too wide to permit selection of liquid asphalts with comparable hardening properties. The behavior of the track section containing sample 1, which immediately developed a hard surface crust, and the slow development of a cohesive surface in the sections containing samples 2, 3, and 6, show the extremes in behavior that may occur.

While the routine laboratory tests indicated that there should be little difference in the road behavior of samples 2, 3, 4, and 6, the behavior of the track sections containing these products definitely show that there are decided differences initially and after weathering in the inherent binding and cementing qualities of these materials that are not indicated by those laboratory tests generally used to determine the conformity to specifications. The raveling that occurred in varying degrees in sections 2, 3, and 6, and the inability of these three sections to resist abrasion under traffic on the alternately wet and dry track, in contrast to the behavior of the section containing sample 4, indicate that the true cementing or binding properties of these materials are not shown by the laboratory tests on the original materials or by stability tests on the mixtures either before or after weathering.

The blended material, sample 6, did not, under the laboratory tests required by the specification for SC–2 material, show any indication of being an unstable blend. Its instability was indicated by the asphaltic residue test run at standard temperature and was definitely shown in the exposure tests and in the behavior of the track section containing this material. It is believed that this material would prove unsatisfactory as a binder in surfacing roads.

Because of the slowness of the sections containing materials 2 and 3 to resist raveling and their inability after considerable aging to resist the scouring action of traffic when wet, it is believed that materials 2 and 3 would not prove entirely satisfactory as binders in wearing surfaces, particularly when used with fine aggregate similar to that used in this investigation.

Materials 1, 4, and 5 produced mixtures that immediately possessed good resistance to the abrasive action of traffic and were not disturbed during the final application of traffic on the alternately wet and dry track. In sections containing materials 1 and 5 some loss of volatile matter and the resultant hardening of the binder might have been responsible for their behavior. The results of tests on sample 4 and mixtures containing it indicate that the loss of volatile matter in the asphaltic material was far too small to account for the early and subsequent behavior of this section.

The relative merits of materials 1, 4, and 5 as binders in this type of construction were not definitely indicated by these tests. The development of cracks in the surface of section 1 could probably have been avoided by additional manipulation of the mixture before spreading.

The more plastic condition of the bituminous mixture in section 1 at the end of the test, in spite of the cracked surface crust, together with the high toughness developed and retained by the cylinders containing sample 1, indicate that this sample is of more durable character than samples 4 and 5 for the following reasons:

1. Sections containing samples 4 and 5 were in a friable condition when disturbed at the end of the test.
2. Impact cylinders containing samples 4 and 5 lost toughness on continued exposure.
3. There was high carbonization in the thin-film exposure residues of samples 4 and 5.

TABLE SHOWING STATUS OF PUBLIC WORKS HIGHWAY CONSTRUCTION OMITTED FROM THIS ISSUE

The table showing the current status of Public Works highway construction has been omitted from this issue. This table will appear in future issues only as space is available but at intervals not exceeding 3 months.

On June 30 Public Works highway funds were involved in 286.8 miles under construction, 117.9 miles were approved for construction and $4,570,020 remained available for new projects.

Copies of the table showing the status by States may be obtained upon application to the Bureau.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JUNE 30, 1937

STATE	1/ APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,872,980	$ 51,600	$ 25,800	9.0	$ 1,354,601	$ 677,300	58.5	$ 4,029,960	$ 2,014,975	177.3	$ 5,154,905
Arizona	5,394,661	2,765,515	2,110,123	151.0	1,332,431	989,780	41.6	697,951	329,004	17.7	1,965,754
Arkansas	6,463,681	65,174	65,029	2.0	3,577,503	3,573,161	212.6	281,712	280,214	36.3	2,545,277
California	14,366,891	9,679,392	5,472,422	269.4	7,815,071	4,243,829	164.4	2,651,589	1,419,031	53.6	3,231,610
Colorado	6,911,198	4,742,008	2,547,978	189.8	3,496,091	1,945,322	119.5	162,993	86,142	3.3	2,331,756
Connecticut	2,388,339	770,265	383,414	14.2	745,158	370,176	8.7	194,520	89,623	1.3	1,545,126
Delaware	1,843,750	745,375	360,062	33.0	415,500	207,664	12.3	340,718	168,337	22.0	1,107,687
Florida	5,020,323	1,240,059	609,422	40.2	1,642,722	821,356	55.9	1,084,970	542,485	11.6	3,047,060
Georgia	9,569,722	2,284,121	1,100,700	139.6	2,763,142	1,381,556	138.8	2,387,926	1,193,963	98.3	5,893,503
Idaho	4,635,991	2,897,006	1,670,144	270.0	1,818,529	1,086,636	117.5	573,983	342,227	42.6	1,536,983
Illinois	15,564,720	9,503,185	4,715,011	145.2	9,149,527	4,524,998	252.9	5,816,578	2,905,428	193.3	3,419,284
Indiana	9,333,269	6,108,500	2,987,215	197.3	6,077,247	3,038,563	165.7	2,084,584	1,042,292	39.1	2,265,198
Iowa	9,757,950	7,436,661	3,663,599	506.2	5,525,075	2,443,447	168.3	1,745,919	845,555	46.9	2,805,349
Kansas	10,005,211	5,066,220	2,528,671	875.5	6,156,702	3,051,745	248.6	1,881,796	940,892	117.2	3,483,904
Kentucky	6,961,271	2,921,522	1,420,361	173.4	2,928,837	1,414,419	63.5	1,983,288	991,754	89.3	3,134,737
Louisiana	5,387,420	2,202,443	1,095,110	81.6	1,208,780	604,080	28.9	9,482,705	1,222,235	46.0	2,465,995
Maine	3,299,867	2,075,544	1,021,149	65.7	1,622,846	811,423	44.1	715,845	357,922	20.3	1,109,373
Maryland	3,094,808				1,620,106	810,020	23.8	497,318	248,659	7.4	2,036,129
Massachusetts	5,255,300	333,935	166,968	3.1	4,434,456	2,217,228	20.3	556,904	278,451	2.3	2,592,653
Michigan	11,562,296	9,944,424	4,867,663	352.3	6,636,412	3,318,206	166.0	2,990,351	1,494,550	70.4	1,881,877
Minnesota	10,344,485	9,109,039	4,322,826	611.7	3,676,462	1,826,980	170.8	712,365	299,314	28.9	3,895,364
Mississippi	6,635,344	4,640	2,320		2,780,210	1,390,050	138.5	2,938,580	1,468,680	134.8	3,774,294
Missouri	11,479,090	5,447,831	2,710,192	498.1	7,865,846	3,907,413	337.0	4,533,708	1,850,427	211.7	3,011,058
Montana	7,744,061	4,297,008	2,314,477	418.4	3,368,773	1,888,515	236.4	1,498,983	781,210	81.4	2,759,860
Nebraska	7,809,353	3,393,372	1,691,588	224.8	4,379,290	2,209,007	477.5	2,157,978	1,047,971	152.0	2,860,787
Nevada	4,821,864	1,709,532	1,452,470	272.9	1,856,559	1,597,569	76.1	798,515	692,427	41.8	1,079,398
New Hampshire	1,843,750	864,727	411,300	25.2	180,047	89,661	1.9	388,810	191,502	5.9	1,151,286
New Jersey	5,054,295	2,709,559	1,336,457	39.3	2,055,959	950,684	20.3	832,890	253,805	1.9	2,513,349
New Mexico	6,030,708	4,187,401	2,567,325	345.7	2,390,381	1,525,658	130.4	1,151,018	701,756	103.9	1,235,969
New York	18,565,567	9,924,332	4,836,779	190.0	17,702,946	8,200,933	290.0	4,559,340	2,256,070	68.7	3,271,785
North Carolina	8,877,837	3,662,221	1,827,476	396.6	5,315,155	2,533,877	333.0	1,576,451	717,845	61.5	3,798,638
North Dakota	5,914,683	359,450	192,450	.4	960,610	940,100	171.4	838,262	838,262	128.9	3,943,871
Ohio	13,771,548	3,449,614	1,708,688	55.2	8,038,411	3,919,692	84.6	2,507,660	1,253,830	37.1	6,889,338
Oklahoma	8,880,547	4,237,711	2,210,200	156.7	2,702,586	1,417,840	99.0	2,572,827	1,331,048	106.9	3,921,459
Oregon	6,182,079	3,205,387	1,937,169	119.5	4,334,036	2,553,062	155.3	528,236	302,243	30.3	1,389,605
Pennsylvania	16,129,804	6,963,896	3,432,354	117.5	11,384,046	5,679,999	160.6	2,882,997	1,437,722	42.0	5,579,729
Rhode Island	1,843,750	284,827	139,343	4.0	1,034,936	517,468	11.9	583,590	290,484	7.4	896,455
South Carolina	5,103,525	561,512	246,900	53.8	5,341,129	2,225,058	328.3	901,891	332,135	42.9	2,299,432
South Dakota	6,162,747	1,400,942	786,366	190.3	1,526,066	857,554	144.2	772,196	430,026	96.7	4,088,801
Tennessee	7,949,380	2,877,307	1,433,351	120.4	2,085,722	1,042,861	72.0	436,740	218,370	16.6	5,254,798
Texas	23,506,431	14,504,926	7,206,201	858.1	13,254,182	6,607,046	836.8	1,486,634	742,618	72.6	8,950,567
Utah	4,274,740	2,412,545	1,688,989	159.7	1,033,158	744,271	92.2	434,570	295,290	42.8	1,546,191
Vermont	1,843,750	1,304,281	637,062	63.0	1,149,866	525,391	32.9	693,611	285,336	16.1	395,960
Virginia	6,887,569	4,087,150	2,060,703	184.2	3,020,189	1,451,896	106.7	904,854	452,427	26.5	2,922,542
Washington	5,907,615	4,795,936	2,482,180	181.6	1,877,590	984,301	63.4	1,550,337	806,798	28.8	1,634,336
West Virginia	4,107,201	863,018	431,486	42.3	1,067,903	533,940	26.6	1,379,037	684,773	40.5	2,457,002
Wisconsin	9,197,557	4,801,159	2,334,736	183.7	6,783,571	2,990,036	204.0	3,429,610	1,522,600	80.5	2,350,186
Wyoming	4,722,322	3,619,047	2,170,902	411.9	2,802,393	1,723,511	280.7	377,620	222,591	42.2	605,318
District of Columbia	1,843,750	29,953	14,542	.8	855,062	420,545	17.2	371,350	182,080	5.4	1,226,583
Hawaii	625,000										625,000
Puerto Rico	368,750,000	175,901,272	91,397,673	9,444.3	191,143,820	98,785,827	7,211.6	83,962,270	38,683,379	2,852.9	139,883,121
TOTALS											

1/ APPORTIONMENTS FOR FISCAL YEARS 1936 TO 1938 INCLUSIVE.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JUNE 30, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,363,985	$ 3,329,855	111.5	$ 690,450	$ 690,450	26.8				$ 130,810
Arizona	2,569,841	3,007,951	2,417,548	188.6	142,121	71,615	7.3				80,678
Arkansas	3,352,061	2,848,120	2,829,570	317.0	466,189	465,015	42.1	$ 16,282	$ 15,714	1.2	41,761
California	7,747,928	6,551,948	6,348,604	238.2	1,596,052	1,393,180	25.7				6,144
Colorado	3,395,263	1,889,718	1,825,329	99.1	89,597	89,596	6.0	8,200	8,200		1,472,138
Connecticut	1,418,709	641,101	590,332	4.7	533,910	516,680	15.7	351,430	261,489	2.2	50,208
Delaware	900,310	605,917	580,185	48.9	247,538	247,538	17.7	37,945	29,151	.2	43,436
Florida	2,597,144	2,048,149	2,014,532	80.7	528,731	528,731	18.3				53,881
Georgia	4,988,967	949,466	939,519	54.2	1,224,868	1,224,868	77.3	1,435,671	1,435,671	77.7	1,388,909
Idaho	2,222,747	2,241,521	2,147,980	185.3	43,487	43,487	.3	9,110	9,110	.3	22,169
Illinois	8,694,009	7,834,755	7,689,701	445.1	869,714	869,636	26.1	16,000	16,000	5.6	118,672
Indiana	4,941,255	3,293,935	3,075,457	138.0	1,849,453	1,849,453	96.8	67,822	9,600	3.6	6,745
Iowa	4,991,664	3,997,213	3,735,369	451.1	1,222,895	1,186,177	76.5	71,113	68,670	.7	1,449
Kansas	4,994,975	4,318,679	4,278,867	344.8	689,173	647,001	31.0	41,632	41,632	14.0	27,476
Kentucky	3,726,271	3,047,002	2,936,633	336.0	571,029	571,029	18.0	165,294	165,294	4.0	53,315
Louisiana	2,890,429	1,869,154	1,687,890	129.8	1,228,623	1,095,927	37.8	112,700	74,701	10.4	31,912
Maine	1,676,799	1,317,092	1,303,342	61.2	348,110	348,110	13.7	25,152	25,152		196
Maryland	1,750,738	475,773	469,405	17.6	747,571	747,571	17.4	193,134	159,302	6.4	374,460
Massachusetts	3,262,885	224,227	224,227	2.7	2,754,490	2,363,720	15.8	1,171,776	603,413	.8	71,525
Michigan	6,301,414	6,440,481	5,942,362	287.8	291,871	291,871	4.8	55,966	44,232	.3	22,949
Minnesota	5,277,145	5,563,716	4,681,207	850.5	795,033	510,207	50.9	99,674	85,731	1.7	
Mississippi	3,457,552	2,457,278	2,453,460	176.1	908,815	907,775	58.6	8,900	8,900	.6	87,417
Missouri	6,012,652	4,525,981	4,483,266	758.1	1,571,541	1,386,824	18.6	37,060	34,964	1.1	107,598
Montana	3,676,416	3,252,638	3,243,064	195.1	380,354	380,354	3.8	85,335	43,996	6.3	9,002
Nebraska	3,870,739	3,075,739	2,977,697	324.0	586,666	586,662	44.0	294,054	294,054	4.5	12,326
Nevada	2,243,074	2,217,596	2,156,630	104.6	86,696	71,696		14,498	14,498	5.2	250
New Hampshire	945,225	615,425	587,476	26.7	280,678	277,700	10.9	81,515	75,283	2.6	4,766
New Jersey	3,129,805	913,747	910,746	16.2	2,141,378	2,128,224	17.9	63,390	63,390	2.0	27,445
New Mexico	2,871,397	2,314,990	2,310,232	179.9	439,817	439,817	23.9	87,831	87,831	8.4	33,517
New York	11,046,377	9,721,035	9,263,466	147.7	1,364,615	1,364,615	22.5	37,477	37,477		380,818
North Carolina	4,720,173	3,063,110	3,030,340	217.1	1,673,500	1,635,700	73.6	1,600	1,600	.1	52,532
North Dakota	2,867,245	2,123,281	2,099,536	316.1	416,525	416,469	42.1	336,821	336,821	58.0	14,420
Ohio	7,670,815	3,665,395	3,595,972	153.4	3,857,197	3,814,762	137.8	145,740	143,220	5.7	116,861
Oklahoma	4,580,670	3,796,397	3,716,098	359.9	678,268	678,208	36.6	248,890	180,440	13.6	5,925
Oregon	3,038,642	2,074,834	2,053,592	154.6	1,202,496	973,279	10.0				11,771
Pennsylvania	9,347,797	2,176,072	2,070,632	104.3	5,264,627	4,907,062	125.5	2,416,905	2,025,796	53.5	344,308
Rhode Island	989,208	1,078,417	983,058	18.8	6,280	3,837		2,710	2,312		
South Carolina	2,702,012	1,779,334	1,673,572	187.1	955,545	887,822	54.4	89,055	89,055	9.6	51,563
South Dakota	2,976,454	2,177,156	2,174,171	409.9	675,137	675,137	71.6	113,508	113,508	14.1	13,638
Tennessee	4,192,460	2,523,809	2,497,017	103.1	1,280,638	1,280,638	37.8	263,660	263,660	7.7	151,146
Texas	11,989,350	12,101,643	11,139,244	1,087.1	738,301	638,575	19.9	258,188	166,102	12.9	45,429
Utah	2,067,154	1,727,883	1,560,426	174.7	433,922	402,158	30.1	70,421	70,421	.6	34,149
Vermont	924,306	986,627	860,168	21.5	81,619	59,280	1.7				4,858
Virginia	3,652,667	3,130,988	3,060,911	941.1	323,866	319,836	82.6	73,506	73,506	1.3	198,414
Washington	3,026,161	3,159,146	2,807,011	163.0	237,855	188,313	1.2				30,836
West Virginia	2,231,412	957,982	954,485	43.6	1,365,248	1,229,349	51.2	57,960	47,577	5.4	
Wisconsin	4,823,884	4,961,434	4,485,554	332.6	368,192	308,219	10.8	22,268	21,900	.3	8,210
Wyoming	2,219,155	2,067,737	2,062,417	139.9	153,801	153,801	12.5				2,937
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	628,269	609,901	8.9	334,873	303,597	8.5				12,535
TOTALS	195,000,000	146,753,846	139,817,552	11,266.7	44,739,355	42,171,571	1,634.1	8,690,193	7,249,373	342.6	5,761,504

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JUNE 30,1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 2,769,103	$ 2,768,839	39	1	12	$ 1,070,215	$ 1,070,215	9			$ 83,900	$ 83,900	1			$ 111,662
Arizona	1,256,099	1,116,158	1,090,424	13			108,151	108,151	1		1	82,734	45,632	1		2	11,892
Arkansas	3,574,060	1,804,637	1,799,310	38	5	2	1,542,218	1,540,068	17	1		212,666	212,396	1		30	22,287
California	7,486,362	5,417,900	5,224,650	31	8		1,993,959	1,949,880	15			311,710	311,710	6			123
Colorado	2,631,567	1,345,381	1,292,033	22	1		898,659	898,658	8			132,720	101,575	2			339,301
Connecticut	1,712,684	141,699	141,699		1		875,781	854,420	6		1	399,790	399,790	2			316,775
Delaware	418,239	130,000	130,000	1								277,993	277,993	2			10,246
Florida	2,827,883	1,757,909	1,755,191	18	5		764,579	763,630	12	1							309,062
Georgia	4,895,949	93,077	91,585	3	2		1,067,054	1,067,054	22	5		755,449	755,449	14	3	52	2,981,861
Idaho	1,674,479	913,521	907,056	14	1	7	621,767	621,654	7	1	9	118,693	118,693	2		5	27,076
Illinois	10,307,184	5,693,389	5,673,397	50	3		4,443,962	4,443,962	23	5							189,825
Indiana	5,111,096	1,967,642	1,819,253	22	11		3,270,149	3,270,149	20	1	163						21,694
Iowa	5,600,679	2,964,073	2,873,798	71	7	5	2,662,446	2,652,027	36	2	2	86,572	74,590			1	264
Kansas	5,246,258	2,778,897	2,774,123	44		5	2,507,540	2,435,294	14	1							36,841
Kentucky	3,672,387	1,083,128	1,077,793	14	3		2,021,533	1,731,801	10	4		846,980	833,388	4	5		29,406
Louisiana	3,213,467	550,142	550,142	7			1,506,423	1,506,422	14	2		1,131,413	928,475	10	1		228,428
Maine	1,426,861	850,767	849,338	15		2	295,326	294,735	5	1		306,173	258,501	2		2	24,288
Maryland	2,061,751	362,915	362,915	3	3	2	513,978	513,978	3		17	920,004	847,536	4	2	3	337,323
Massachusetts	4,210,833	1,231,713	1,231,628	10	2		2,186,644	2,183,064	14	2		739,651	739,651	3			56,490
Michigan	6,765,197	4,612,994	4,524,126	38	5		2,267,927	2,197,458	6	3		43,500	43,500		1		113
Minnesota	5,395,441	3,789,003	3,715,881	70	11	36	1,643,989	1,617,990	15	2	3	57,010	57,010			11	4,560
Mississippi	3,241,475	1,206,553	1,206,389	33	3	1	1,466,531	1,466,531	22	3	1	89,800	89,800	1		13	478,754
Missouri	6,142,153	823,705	822,880	15			5,485,281	5,289,037	34	1	1	1,650	1,650			1	28,585
Montana	2,722,327	2,579,716	2,458,111	36	7		118,967	118,967	1			198,775	145,096	1	1		153
Nebraska	3,556,441	1,934,181	1,909,203	67	2	8	1,433,416	1,433,416	15	1	5	93,477	93,477	4		11	120,345
Nevada	887,260	688,420	682,637	7	3		211,658	193,432	1			3,630	3,630			5	7,561
New Hampshire	822,484	341,748	341,748	3	3		395,145	395,100	5	1	1	68,825	68,803	1		6	16,833
New Jersey	3,983,826	893,256	893,256	7	2	1	2,421,099	2,410,054	14	5		743,310	680,515	3	3		
New Mexico	1,725,286	1,039,545	1,038,317	13	1		673,090	670,400	5			56,505	11,202	1			5,367
New York	13,577,189	5,228,530	5,168,806	22	22		8,351,500	8,136,700	25	26		94,000	94,000				177,683
North Carolina	4,823,958	1,944,493	1,944,493	25	15		2,032,307	2,012,181	23	3	108	54,950	54,950	1			812,335
North Dakota	3,207,473	1,404,831	1,402,057	26	2		1,465,516	1,464,516	25	2		340,900	340,900	1			
Ohio	8,439,897	481,176	415,684	3	1	2	5,920,836	5,748,289	39	7	1	2,133,490	1,835,000	14			440,925
Oklahoma	5,004,711	2,724,357	2,715,710	44	4	2	947,328	947,328	12	4	3	1,456,763	1,323,663	9	4	36	18,010
Oregon	2,334,204	1,193,881	1,187,909	10	5	2	1,170,816	1,089,215	6	1		51,387	51,387	1			5,693
Pennsylvania	11,483,613	2,350,848	2,104,071	38	9	9	8,344,686	7,718,326	39	11		1,594,956	1,558,711	15			102,505
Rhode Island	699,691	649,511	648,445	4	2		36,240	36,240		1		12,567	12,567				2,439
South Carolina	3,059,956	1,114,398	1,103,681	22	6	3	1,159,735	1,151,601	21	6	17	323,328	301,528		4	28	503,145
South Dakota	3,249,086	1,132,083	1,131,486	28	3	25	1,850,032	1,850,032	35	3	19	321,170	262,212	13		9	5,356
Tennessee	3,903,979	775,083	767,048	15	2	19	2,546,860	2,546,860	29	1	3	232,510	232,510	2		15	357,561
Texas	10,855,982	7,427,926	7,416,645	104	13	37	2,641,561	2,644,558	26	1	10	411,378	411,377		1	111	383,402
Utah	1,230,763	343,803	342,397	3	1	1	873,921	873,921	14								14,444
Vermont	729,857	497,840	486,213	7	5	15	1,271,833	243,644	3	2	5						
Virginia	3,774,287	2,053,045	1,943,175	35	12	9	1,001,516	1,001,516	10	5	2	754,414	746,692	6	2	11	82,903
Washington	3,095,041	2,231,646	2,196,919	20	10	8	883,510	883,085	3	2	1	4,562	4,562			2	10,476
West Virginia	2,677,937	79,454	79,454	1		4	1,823,821	1,821,853	17	3		695,138	695,138	8	1	1	81,492
Wisconsin	5,022,683	2,579,025	2,541,335	27	4		2,455,480	2,455,436	10	3		23,268	22,867			5	44
Wyoming	1,260,841	665,281	665,173	9			617,516	617,515	4			18,153	18,153	1			
Dist. of Columbia	410,804	425,564	396,804	3													14,000
Hawaii	453,703	170,404	170,389	2			351,976	283,314	3								
TOTALS	196,000,000	86,354,351	84,836,616	1152	206	217	90,217,477	87,223,677	698	123	373	16,345,864	15,210,179	136	28	360	8,729,528

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 6 AUGUST 1937

THE BROAD HIGHWAY BECKONS TO MOTOR VAGABONDS

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

Volume 18, No. 6 August 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

TOURIST TRAVEL IN THE UNITED STATES

A SUMMARY OF AVAILABLE DATA ON HIGHWAY USE BY TOURISTS

BY THE DIVISION OF HIGHWAY TRANSPORT, BUREAU OF PUBLIC ROADS

Reported by L. E. PEABODY, Senior Highway Economist, and I. MANSFIELD SPASOFF, Assistant Highway Economist

Photo Copyrighted by Asahel Curtis

MAJESTIC MOUNTAINS MIRRORED IN PLACID LAKES LURE TOURISTS TO MOUNT RANIER NATIONAL PARK, WASHINGTON.

IN RECENT years Americans have become more and more interested in visiting the historical and scenic attractions of the United States. With the improvement of each additional mile of highway, with the acquisition of each additional passenger car, and with the increase in personal income during the recent years of returning prosperity, ever-increasing numbers of tourists are traveling from end to end of the country. At the height of prosperity, estimates by investigators placed the value of American tourist business at several billion dollars annually.

In the wake of the downward plunge into depression, the paralysis of industry, the shrinking of values, and the reduction of income, the tourist business necessarily declined. The falling-off in tourist travel became most acute between 1930 and 1932. But this remarkable movement, which went down with the tide of prosperity, appeared again on the crest of the first rising wave, for in 1934 total tourist expenditures in the United States were again estimated at several billion dollars. Estimates of trends in tourist travel and of the expenditures by tourists for the country as a whole are necessarily based on incomplete data since comprehensive records are not available.

The value of tourist trade has been estimated for a number of States. Maine ranks it second only to her entire agricultural output; California places it next in importance to her great petroleum industry; in Michigan, the center of the automotive industry, it ranks second; in Wisconsin its value as a producer of revenue is exceeded only by that of the dairy industry; and in Florida, during the 1935–36 season, it is estimated to have represented many times the value of the entire citrus fruit crop. From practically every State there

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comes the story of a large increase in expenditures by tourists.

Throughout the United States the various means of transportation are developed to a high degree. One may go with facility by boat, train, automobile, or airplane from New York to San Francisco, the chief factors determining the choice being personal preference, expense, or time. Occasional estimates have been made of the extent to which each of these modes of travel is used by the American tourist, but few figures are available for determining a progressive trend over a series of years. It is certainly true, however, that the great majority of American tourists who are out to see their own country travel either by rail or automobile. Figures by the National Park Service of the United States Department of the Interior, relating to the number of visitors arriving by rail and by automobile at Yellowstone National Park each year from 1922 to 1930, inclusive, give an interesting glimpse of the trend in these modes of travel (see table 1).

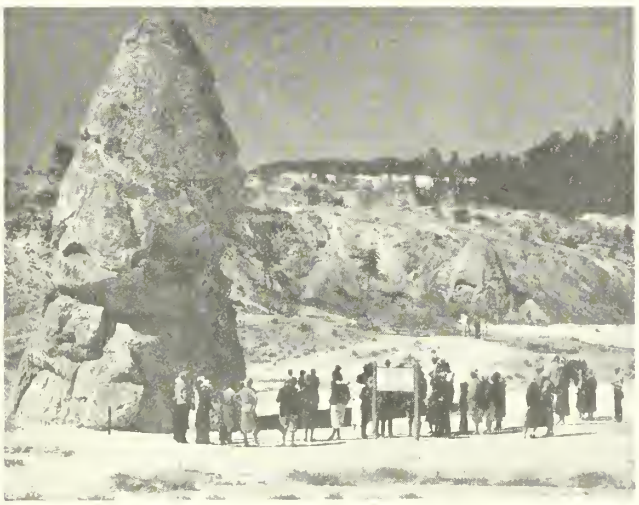


Photo by Courtesy of the United States Department of the Interior

YELLOWSTONE NATIONAL PARK IS ANNUALLY VISITED BY THOUSANDS OF TOURISTS FROM ALL PARTS OF THE COUNTRY. LIBERTY CAP, SHOWN ABOVE, IS BUT ONE OF A WIDE VARIETY OF ATTRACTIONS THAT INCLUDE GEYSERS, WATERFALLS, AND TOWERING MOUNTAINS.

LARGE MAJORITY OF VISITORS TO NATIONAL PARKS TRAVEL BY AUTOMOBILE

Figures from the same source show that during those years the number of visitors arriving by rail and automobile represented a fairly constant proportion of approximately 90 percent of all visitors to the park, with about 10 percent arriving by all other modes of transportation. Of the combined number of visitors arriving by rail and automobile, presented in table 1, an increasing trend is shown in the number arriving by automobile.

In 1922 the percentage of visitors arriving by rail was 33.7 percent, and that of visitors arriving by automobile was 66.3 percent of the combined total. With slight variations in the rates of change during subsequent years, the proportion of visitors arriving by automobile increased to 88.6 percent in 1930, while that of visitors arriving by rail declined to 11.4 percent. Comparable figures are not available for the years 1931 to 1935, but an estimate for 1936 indicates a continuation of the previous trend. This estimate shows that the number of visitors arriving by rail had remained about constant at 22,000, while the number arriving by automobile had increased to 370,000, representing 5.6 and 94.4 percent, respectively, of the combined total.

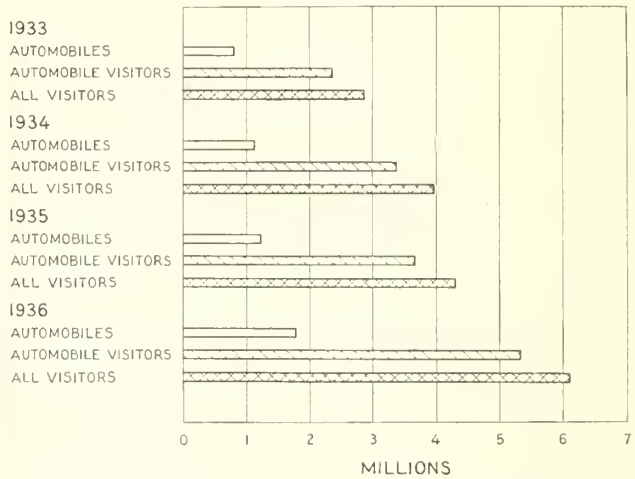


FIGURE 1.—NUMBER OF AUTOMOBILES AND VISITORS ENTERING ALL NATIONAL PARKS, 1933–36, INCLUSIVE.

TABLE 1.—*Visitors to Yellowstone National Park from the continental United States, arriving by rail and by automobile 1922–30* [1]

Year	Visitors arriving by rail		Visitors arriving by automobile		Visitors arriving by rail and automobile	
	Number	*Percent*	*Number*	*Percent*	*Number*	*Percent*
1922	29,329	33.7	57,775	66.3	87,104	100.0
1923	34,781	28.5	87,286	71.5	122,067	100.0
1924	35,846	27.0	96,884	73.0	132,730	100.0
1925	39,940	28.6	99,881	71.4	139,821	100.0
1926	32,477	19.8	131,895	80.2	164,372	100.0
1927	38,811	21.3	143,200	78.7	182,011	100.0
1928	35,262	17.2	169,436	82.8	204,698	100.0
1929	34,789	15.0	197,032	85.0	231,821	100.0
1930	22,759	11.4	176,910	88.6	199,669	100.0

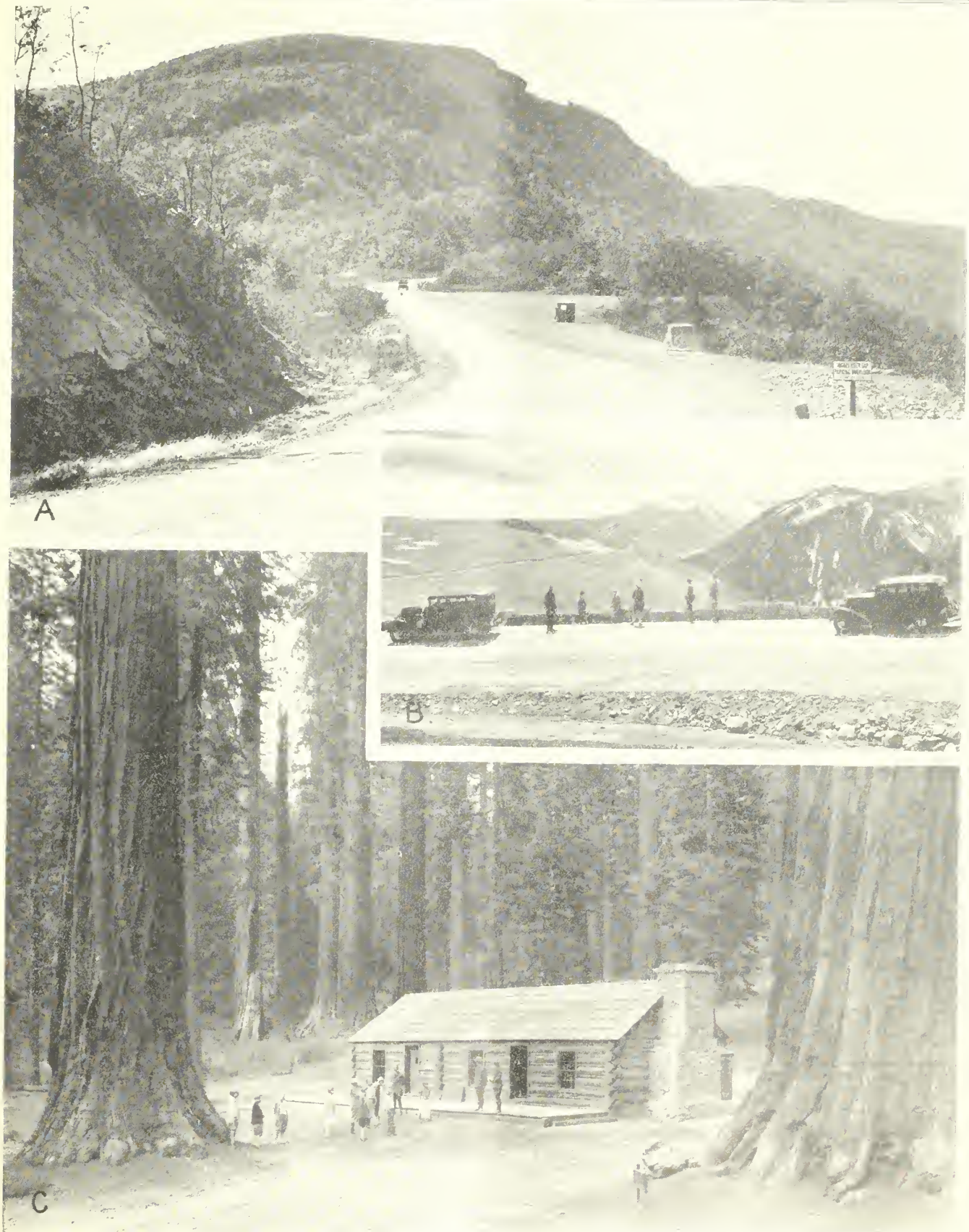
[1] Data compiled from releases of the National Park Service, United States Department of the Interior, and presented in Communication Agencies and Social Life by Willey and Rice; Recent Social Trends Monograph Series.

The years 1932 and 1933 witnessed a decline in tourist traffic in all parts of the United States, but this falling-off was followed by a marked increase in volume in many parts of the country during 1934, with continuation of the upward trend in 1935 and 1936. Data showing the total number of automobiles, automobile passengers, and all visitors, entering all national parks during the past 4 years, are presented in table 2 and are shown graphically in figure 1. A gradual increase in the number and percentage of visitors arriving by automobile is shown, but in 1935 the increase over 1934 was very slight, as was also the

TABLE 2.—*Total number of automobiles, automobile passengers, and all visitors, entering all national parks, 1933–36* [1]

Year	Cars	Automobile visitors		All visitors	
		Persons [2]	Increase	Number	Increase
			Percent		*Percent*
1933	788,809	2,366,427		2,867,374	
1934	1,124,586	3,373,758	42.6	3,965,720	38.3
1935	1,217,054	3,651,162	8.2	4,284,615	8.0
1936	1,772,538	5,317,014	45.6	6,082,081	42.0

[1] Basic data supplied by National Park Service, U. S. Department of the Interior.
[2] Estimated on the basis of an average of 3 passengers to a car.



Photos by Courtesy of the United States Department of the Interior

VIEWS IN THREE NATIONAL PARKS IN DIFFERENT PARTS OF THE COUNTRY THAT ATTRACT TOURISTS FROM FAR DISTANT POINTS. A, THE SKYLINE DRIVE, IN THE SHENANDOAH NATIONAL PARK, WILL EVENTUALLY EXTEND SOUTH AND WEST INTO THE GREAT SMOKY MOUNTAINS NATIONAL PARK OF NORTH CAROLINA AND TENNESSEE; B, TRAIL RIDGE ROAD WITH LONG'S PEAK IN THE DISTANCE IN ROCKY MOUNTAIN NATIONAL PARK, COLORADO; C, GIANT REDWOOD TREES IN YOSEMITE NATIONAL PARK, CALIFORNIA.

ease with regard to all visitors. Judging from fragmentary records, estimates, and news items, the trend of tourist activity seems to have been approximately the same in various resort areas, with the definite improvement in 1934 being sustained in succeeding years.

NEED FOR ACCURATE DATA ON TOURIST TRAFFIC RECOGNIZED

Up to a comparatively recent date, measures of the actual volume and value of annual tourist traffic were largely conjectural. There was comparatively little direct information available for making such estimates. The very difficulty of making reasonable estimates, however, and the stimulating effect of competition among the individual States for a share in the motor tourist business resulted in a more systematic effort to obtain accurate basic information. Automobile clubs and tourist organizations have been active in collecting this type of information from their contacts in many localities. An estimate made by the American Automobile Association of the geographic distribution of motor tourist traffic in 1928, one of the biggest tourist years, is the only one of its kind that has been discovered in the course of considerable search. According to this estimate, the distribution of motor tourist traffic among the various resort areas during that year, was as follows:

	Percent
Far West, Great Lakes, and Southwest	59
Northeast	11
Central Appalachian	9
Northwest	7
Southeast	5
Gulf and Central	2
Observed in home State	7

Through their State highway departments, or through organizations sponsored by various business interests, many States have started to compile specific data relating to their tourist traffic. At regular intervals since 1923, at 33 points along the State line, the Wisconsin Highway Commission has conducted a traffic census of out-of-State cars entering and leaving the State during the 100 days between June 1 and Labor Day. A questionnaire card, to be filled out and returned by each driver, covers the length and cost of his visit, the distance traveled, and the reason for his trip.

At Bath, Maine, a record has been kept for years of all highway traffic crossing the Kennebec River on the only direct highway from Portland and points south to resorts along the upper Maine coast. A State development commission was created by the New Hampshire Legislature in 1925. The New England Council, also organized in 1925, has since then coordinated the work of the six New England States, with a view to developing and publicizing their recreational advantages. This organization was originally sponsored by the business interests of the several States; but in 1935, through official action of the legislatures and governors, public funds were made available to carry on the work. During that same year (1935) the Governor of New York signed a bill appropriating money for the establishment of a publicity bureau in that State.

In California, both the department of motor vehicles and the department of agriculture have made counts of cars of out-of-State registration entering the State. Analytical studies of tourist traffic to California have also been published by Californians, Inc., and by the All-Year-Club of Southern California. The Colorado Association has made similar studies of tourist traffic in that State.

Within recent years, surveys of the movement of motor traffic upon their highways have been made by the highway departments of a number of States, independently or in cooperation with the Bureau of Public Roads. In the earliest of these, very little information relating to tourist traffic was obtained. The most that can be found in some of the published reports is the volume or relative importance of out-of-State passenger-car traffic. But the later studies present a more detailed analysis of the characteristics of automobile tourist traffic.

The reports of surveys that were made in a number of Northeastern States between 1924 and 1927 showed the percentage of out-of-State cars on the State highways during the summer months, as follows: New Hampshire, 51.1 percent; Vermont, 36.6 percent; Maine, 23.3 percent; Connecticut, 21.1 percent; Pennsylvania, 14.0 percent; and Ohio, 10.2 percent. The actual volume of traffic indicated by these percentages depends upon the total amount of traffic within each State. Therefore, such percentages should not be taken as an index of the relative importance of tourist traffic visiting these States. For example, insofar as motor-vehicle registration can be taken as a rough measure of relative State traffic, 51 percent of all passenger vehicles registered in New Hampshire would be less than one-third of the number represented by 10 percent of passenger-vehicle registrations in Ohio. The percentages given are of value only in relation to traffic within the individual State and have no significance from a national point of view.

TOURIST FIGURES AVAILABLE FOR 11 WESTERN STATES AND MICHIGAN, FLORIDA, AND ARKANSAS

The survey of traffic on the Federal-aid highway systems of 11 Western States, made between September 1929 and October 1930, revealed figures showing for each State the number of visiting cars registered in each of the 11 States and in other areas throughout the Nation. The average daily mileage traveled by out-of-State cars was also recorded. No information regarding expenditures by these quasi-tourist groups was obtained in connection with that survey, however.

A similar survey of highway transportation in the State of Michigan was made between July 1930 and July 1931. By means of questionnaire cards, information was obtained regarding the place of origin, number of persons in the party, duration of the visit, trip mileage within the State, and type of accommodation used. Based on information from various sources, such as tourist associations and automobile clubs, an estimate of expenditures was also made in connection with this survey.

In a general survey of traffic in the State of Florida, made between October 1933 and October 1934, the information obtained in regard to out-of-State passenger car traffic was the same as that recorded in the Michigan survey except that the questionnaire card contained an additional inquiry regarding the specific purpose of the visit. Based on average figures reported by a large number of separate parties of various types, estimates of total and average expenditures were also made.

The tourist questionnaire card used in connection with the Arkansas traffic survey, conducted between April 1934 and June 1935, was similar to that used in the Florida survey, except that the purpose of the visit was limited to the simple designation of

"business or pleasure", instead of the longer enumeration used on the Florida questionnaire card. No estimate of expenditures was made in connection with this survey.

Thus, it will be seen that data are available, in the reports of the highway surveys mentioned, regarding the place of origin and average daily mileage traveled by out-of-State automobile traffic in 11 western States, viz, Arizona, California, Colorado, Idaho, Nebraska, Nevada, New Mexico, Oregon, Utah, Washington, and Wyoming, and in Michigan and Florida. Data relating to the origin of out-of-State cars are also available in the Arkansas report, but trip mileage figures are not shown for that State. The number of persons per car, duration of visit in days, and type of accommodation used are recorded for Michigan, Florida, and Arkansas; estimates of expenditures are included in the Michigan and Florida reports; and the purpose of the visit is given in those of Florida and Arkansas. Similar information compiled in connection with studies of tourist traffic made by governmental or other organizations, is also of interest in a discussion of these several characteristics of American tourist traffic.

In the various traffic reports already described, the term "tourist" is used to describe traffic originating outside of the State and visiting the State for more than 1 day. It is not limited to those traveling for recreation, and includes those making business trips. In the more recent surveys in Florida and Michigan, persons visiting the State for 1 day or less were not classified as tourists, while in Arkansas information was obtained regarding the number of cars in the State on business, and details relating to cars passing through the State en route to other States were shown in certain tabulations.

Photo by California State Highway Department

TOURIST TRAFFIC IS ENCOURAGED AND THE TOURIST SEASON IS LENGTHENED IN MANY STATES BY SNOW REMOVAL ON THE MAIN HIGHWAYS.

WEALTHY, DENSELY POPULATED AREAS PROLIFIC SOURCES OF AUTOMOBILE TOURIST TRAFFIC

The determining factors reflected in the volume of out-of-State passenger car traffic are manifold. One set of factors exerts its influence from the point of origin of such traffic, and another from the point of destination. The former acts as a centrifugal force, tending to drive traffic out from a certain point; the other acts as a centripetal force, exerting a pull toward another point. Included in the first set of factors are actual population and population density per square mile,

together with passenger-car registration and reported total income. The volume of outgoing tourist traffic may be expected to vary directly with the magnitude of these several influences. The second set involves the extent and perfection of highway development in the area or State of destination; the less tangible factors represented by climatic, recreational, health, or scenic attractions; and the extent of influence of all these through direct knowledge, hearsay, or publicity.

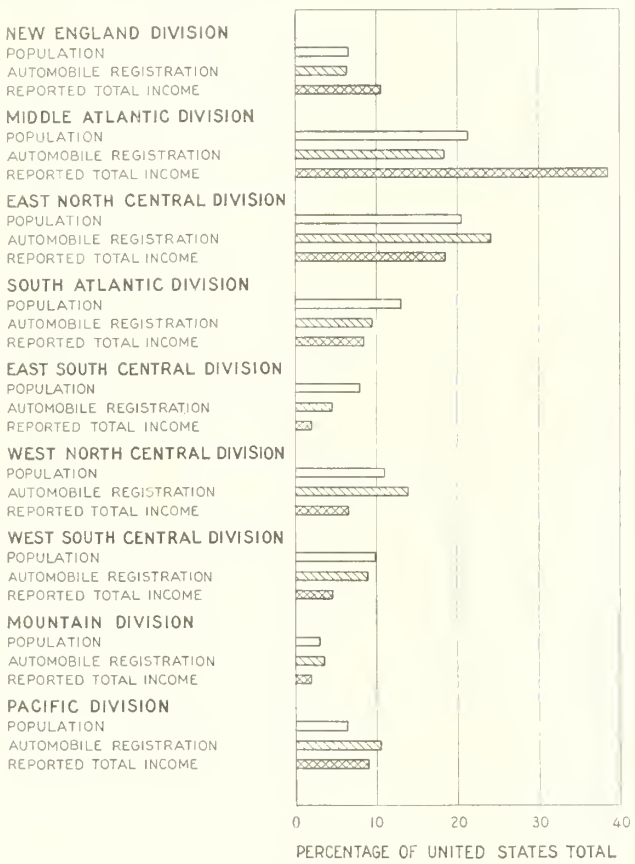


FIGURE 2.—RELATIVE IMPORTANCE OF EACH GEOGRAPHIC DIVISION IN VARIOUS FACTORS AFFECTING ORIGIN OF AUTOMOBILE TOURIST TRAFFIC.

Yet another factor, which exerts a tremendous influence either as an attraction or as a repellent, is distance. The proximity of certain points of interest to areas of origin tends to attract tourist traffic which would be unlikely to visit similar points at a greater distance. Thus, the factor of distance may be said to work inversely, all other things being equal, i. e., the greater the distance between origin and destination, the less the volume of automobile tourist traffic.

Varying amounts of tourist travel (as defined in the traffic reports by the Bureau) result from the normal transaction of business. Although short trips of less than a day's duration may be eliminated in the traffic survey reports, there are many who remain in the State for more than 1 day on business trips. Business activity within the State and the nearness of large cities beyond the State borders are important factors. The size of the State and the extent to which it is traversed by through highways carrying traffic destined primarily for other States are also important. A centrally located State may have much traffic consisting of cars making business trips.

The relative importance of various sections of the United States as potential sources of automobile tourist traffic is indicated by the figures in tables 3 and 4, and the principal data are also shown graphically in figure 2. As in almost every other line of business activity, the centers of greatest population offer the most promising market for the "sale" of tourist trips. Both the greatest actual population and the greatest density per square mile are found in the Middle Atlantic States—New York, New Jersey, and Pennsylvania. Next in importance in actual population are the East North Central States—Ohio, Indiana, Illinois, Michigan, and Wisconsin—though the population density is less in this area than in New England. The Middle Atlantic and East North Central States also show the greatest percentages of passenger-vehicle registrations, as well as of total income reported on income tax returns. The concentration of these significant factors in the States east of the Mississippi, and north of the Ohio River, justifies the assumption that this area should contribute more tourists to the great annual migration than any other part of the country.

TABLE 3.—*Population and passenger-vehicle registration by geographic divisions and sections, 1936*

Geographic division,[1] and section	Population			Passenger vehicle registration[3]	
	Number[2]	Percentage of United States total	Per square mile	Number	Percentage of United States total
	1,000 persons			*1,000 vehicles*	
New England	8,581	6.7	138.5	1,509	6.2
Middle Atlantic	27,399	21.3	274.0	4,578	18.9
East North Central	25,708	20.0	104.7	5,752	23.8
South Atlantic	17,072	13.3	63.4	2,459	10.2
East South Central	10,619	8.3	59.2	1,052	4.4
East of the Mississippi River	89,379	69.6	104.4	15,350	63.5
West North Central	13,782	10.7	27.0	3,134	13.0
West South Central	12,790	10.0	29.8	2,028	8.4
Mountain	3,759	2.9	4.4	887	3.7
Pacific	8,719	6.8	27.4	2,757	11.4
West of the Mississippi River	39,050	30.4	18.4	8,806	36.5
United States total	128,429	100.0	43.2	24,157	100.0

[1] The classification used is that of the Bureau of the Census, as follows: New England—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut; Middle Atlantic—New York, New Jersey, Pennsylvania; East North Central—Ohio, Indiana, Illinois, Michigan, Wisconsin; South Atlantic—Delaware, Maryland, District of Columbia, West Virginia, Virginia, North Carolina, South Carolina, Georgia, Florida; East South Central—Kentucky, Tennessee, Alabama, Mississippi; West North Central—Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas; West South Central—Arkansas, Louisiana, Oklahoma, Texas; Mountain—Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada; Pacific—Washington, Oregon, California.

[2] Midyear estimates of the U. S. Bureau of the Census.

[3] Bureau of Public Roads figures, compiled from reports by State authorities. For the majority of States the registration of private passenger cars is not shown separately; but for those in which this distinction is made, private cars constitute about 99 percent of total passenger-vehicle registration.

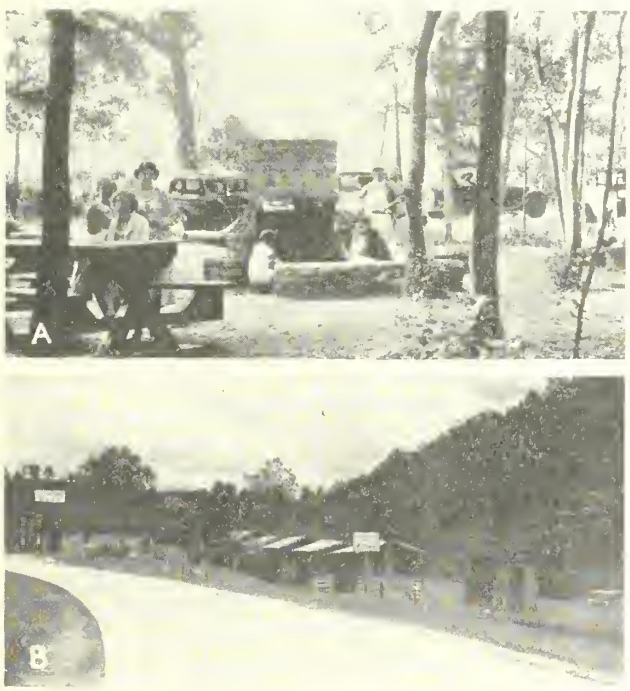
EASTERN STATES HAVE ONE-HALF OF THE SURFACED ROADS BUT ONLY ONE-FOURTH OF THE LAND AREA OF THE UNITED STATES

More than half of the surfaced mileage of State highway systems is found east of the Mississippi River, where less than 29 percent of the total land area is located; and more than 31 percent of this class of roads is in the northeastern States which comprise less than 14 percent of the total land area of the United States. Therefore, it may be assumed that the States east of the Mississippi River, especially the northeastern group, are the most fruitful field of origin of tourist traffic and are also outstanding in highway facilities to at-

TABLE 4.—*Individual income reported on income tax returns by geographic divisions and sections, 1932* [1]

Geographic division and section [2]	Income-tax returns		Total reported income			Income other than earned and business	
	Number	Percentage of population [3]	Amount	Percentage of United States total	Average income	Amount	Percentage of United States total
			1,000 dollars		*Dollars*	*1,000 dollars*	
New England	413,699	5.0	1,500,276	10.5	3,626	552,095	12.4
Middle Atlantic	1,350,968	5.1	5,506,347	38.4	4,076	1,940,787	43.5
East North Central	757,823	2.9	2,685,266	18.7	3,543	690,856	15.4
South Atlantic	335,461	2.1	1,235,522	8.6	3,683	366,262	8.2
East South Central	95,225	1.0	322,615	2.2	3,388	79,965	1.8
East of the Mississippi River	2,953,176	3.4	11,250,026	78.4	3,809	3,629,965	81.3
West North Central	272,633	2.0	931,102	6.5	3,415	234,941	5.3
West South Central	179,122	1.4	629,869	4.4	3,516	167,750	3.7
Mountain	82,378	2.2	263,324	1.8	3,197	67,453	1.5
Pacific	377,929	4.4	1,272,966	8.9	3,368	364,127	8.2
West of the Mississippi River	912,062	2.4	3,097,261	21.6	3,396	834,271	18.7
United States total	3,865,238	3.1	14,347,287	100.0	3,712	4,464,236	100.0

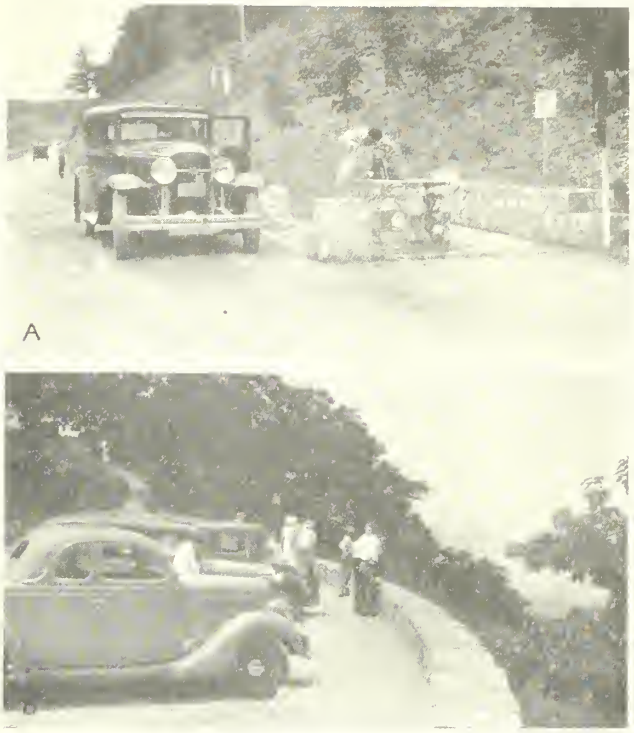
[1] Figures are from Statistics of Income, Report of the Commissioner of Internal Revenue, Treasury Department, and are for the continental United States, not including Hawaii.
[2] See note 1 table 3.
[3] Based on midyear estimates of population in 1932.



Upper Photo by Rhode Island State Highway Department

ROADSIDE FACILITIES AWAIT USE BY THE TIRED AND HUNGRY TOURIST. A, FULLY EQUIPPED PICNIC AREA IN RHODE ISLAND. B, ROADSIDE CABIN CAMP IN WEST VIRGINIA.

tract such traffic. The greater distances found in the central plains and Western States may be expected to act as a restraint upon the interchange of tourist traffic between eastern and western parts of the country. The figures in table 5 show the actual mileage and distribution of surfaced highways in each geographic division, and the relative importance of such highways in relation to total land area.



A, ROADSIDE DRINKING FOUNTAINS, SUCH AS THIS ONE IN WEST VIRGINIA, INVITE THE THIRSTY TOURIST TO STOP AND REFRESH HIMSELF. B, THIS OVERLOOK ON THE SKYLINE DRIVE IN VIRGINIA IS TYPICAL OF MANY SUCH ACCOMMODATIONS FOR MOTORISTS.

It would be impossible to enumerate the points of interest in every part of the country which may be credited with different degrees of influence in attracting tourists to individual States. New England and the Great Lakes region have their cool summer climate, and Florida her warm winter sun; the variety of both its climate and scenery recommend California as an all-year resort and playground; and national parks beckon from far and near to lovers of nature throughout the land. The reasons for individual choice of a vacation's locale are of almost infinite variety.

TABLE 5.—*Surfaced mileage on primary State highway systems* [1] *and land area per mile of such highways, by geographic divisions and sections, 1935*

Geographic division [2] and section	Surfaced State highways		Land area, square miles	
	Number of miles	Percentage of United States total	Total	Per mile of surfaced State highway
New England	10,807	3.7	61,976	5.7
Middle Atlantic	28,635	9.7	100,000	3.5
East North Central	52,291	17.8	245,564	4.7
South Atlantic [3]	46,821	15.9	269,011	5.7
East South Central	22,463	7.6	179,509	8.0
East of the Mississippi River	161,017	54.7	856,060	5.3
West North Central	53,647	18.2	510,804	9.5
West South Central	33,793	11.5	429,746	12.7
Mountain	25,449	8.7	859,009	33.7
Pacific	20,274	6.9	318,095	15.7
West of the Mississippi River	133,163	45.3	2,117,654	15.9
United States total	294,180	100.0	2,973,714	10.1

[1] Includes urban extensions on designated State systems.
[2] States included in the respective geographic divisions are listed in note 1 of table 3.
[3] Does not include the District of Columbia.

The amount of publicity given certain areas through Nation-wide or local advertising campaigns is a factor of no small importance. According to testimony presented at a hearing before a subcommittee on interstate and foreign commerce of the United States House of Representatives: "It has been estimated that in 1930, $50,000,000 was spent in advertising travel objectives, the funds being provided by transportation agencies, States, local communities, hotels, regional associations, and various business interests."

It is interesting to note, in this connection, the experience of the New England Council during 4 years of systematic advertising. The primary circulation of its advertising media during this period was 120,000,000. As a result of this advertising 125,000 inquiries were received, representing the awakening of active interest in approximately one per thousand, or about one-tenth of 1 percent, of the possible prospects. It is impossible to determine how many of these inquirers actually made trips to New England; and it is also impossible to estimate how many made trips as a result of such advertising.

There are still other occasional or accidental factors that influence the distribution of tourist traffic. Local fairs or expositions, or the opening of national park or scenic areas, hitherto inaccessible to large numbers of tourists, may cause a great temporary influx of tourist traffic, which will continue in much smaller volume after the novelty has passed. Another factor is the location of features of comparatively minor interest which would attract few visitors except for their nearness to the line of travel to other important tourist meccas.

VOLUME OF TOURIST TRAFFIC FOUND TO VARY INVERSELY WITH DISTANCE OF TRAVEL

The importance of distance in determining the proportion of tourist automobile traffic originating in various parts of the country is illustrated in the figures for 11 Western States, and also in those for Michigan, Florida, and Arkansas. Three of the 11 Western States participating in the traffic survey of 1929–30 are on the Pacific coast, 7 are Mountain States, and Nebraska is one of the most westerly of the Central Plains States.

When the figures reported for out-of-State passenger cars in these States are combined into sectional groups, as shown in table 6, the effect of distance may be readily seen. The Pacific States received only 12.4 percent of their out-of-State traffic from States east of the Mississippi River, while the Mountain States received 15.6 percent from that area, and Nebraska 20.1 percent. The Pacific States, because of their accessibility to visitors from Mexico and Canada, had relatively more traffic from foreign countries than did the Mountain States or Nebraska. Including this foreign passenger-car traffic, the percentage of out-of-State traffic from points west of the Mississippi River was complementary to the figures listed above, being 87.6 percent for the Pacific States, 84.4 percent for the Mountain States, and 79.9 percent for Nebraska.

In addition to this tendency for the amount of out-of-State passenger-car traffic to vary inversely with the distance of travel, a comparison of the percentages for the States in each group illustrates the strength of the tourist appeal of particular areas. In the Pacific group, for example, California drew 25.2 percent of its tourist traffic from east of the Mississippi River, in contrast to only 6 percent for Washington and Oregon combined. Similarly, five mountain States of major tourist interest received 17.8 percent of their tourist traffic from that area, in comparison with 9.9 percent for Nevada and Idaho. Corresponding figures for individual States are presented in table 7.

TABLE 6.—*Origin of out-of-State passenger car traffic in 11 Western States, 1929–30; percentage distribution*

Area of origin [1]	Traffic in Pacific States			Traffic in Mountain States			Traffic in Nebraska
	All States	California	2 States [2]	All States [3]	5 States [4]	2 States [5]	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New England States	1.0	2.0	0.5	1.0	1.1	0.8	0.9
Middle Atlantic and East North Central States [6]	10.2	20.9	4.9	13.1	15.0	8.2	18.2
South Atlantic and East South Central States [7]	1.2	2.3	.6	1.5	1.7	.9	1.0
East of the Mississippi River	12.4	25.2	6.0	15.6	17.8	9.9	20.1
West North Central and West South Central States [8]	10.1	17.4	6.4	28.2	35.0	11.5	51.5
Mountain States [3]	14.4	18.3	12.4	21.0	20.3	22.7	17.6
Pacific States	57.7	35.1	69.0	34.5	26.3	55.0	10.4
West of the Mississippi River	82.2	70.8	87.8	83.7	81.6	89.2	79.5
United States, total	94.6	96.0	93.8	99.3	99.4	99.1	99.6
Foreign countries	5.4	4.0	6.2	.7	.6	.9	.4
Grand total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Except as noted, States are grouped according to the census classification. See note 1 of table 3.
[2] Washington and Oregon.
[3] Except Montana.
[4] Wyoming, Colorado, New Mexico, Arizona, and Utah.
[5] Idaho and Nevada.
[6] Includes Delaware and Maryland.
[7] Does not include Delaware and Maryland.
[8] Includes Montana.

EIGHTY PERCENT OF MICHIGAN'S TOURIST TRAFFIC CAME FROM FOUR NEIGHBORING STATES

The influence of distance on the amount of out-of-State passenger-car traffic is also clearly illustrated by the figures in table 8, which are derived from traffic reports for Michigan, Florida, and Arkansas. As has already been pointed out, the East North Central States are among the most important in the principal factors contributing to the creation of considerable tourist traffic. Michigan is one of these States, and may be expected to draw heavily upon this reservoir of potential tourist traffic because of its proximity. The estimated total number of out-of-State cars visiting Michigan during the period of the traffic survey was 2,500,000 annually, which is equal to nearly 11 percent of all passenger vehicles registered outside the State of Michigan during 1930. For no other State except Wisconsin, which enjoys advantages of climate and location similar to those of Michigan, have figures been found which approach that volume of tourist car traffic, the number of visiting cars reported for Wisconsin being 1,902,500 in 1931.

Michigan's tourist appeal is recorded not only in the volume, but also in the widespread origin of its tourist traffic. A news item published in June 1935 stated that, among visitors who registered at a tourist lodge and clearing-house of information during the month after its establishment near the Indiana border, there were persons from 42 States and 2 foreign countries. The distribution according to point of origin of passenger cars visiting Michigan, recorded in the survey of

TABLE 7.—*Origin of out-of-State passenger car traffic in each of 11 Western States, 1929–30, percentage distribution*

Area of origin [1]	Traffic in Pacific States			Traffic in Mountain States [2]						Traffic in Nebraska
	Washington	Oregon	California	Idaho	Wyoming	Colorado	New Mexico	Arizona	Utah	Nevada	
New England States	0.5	0.4	2.0	0.6	1.0	1.1	0.9	1.2	1.2	0.9	0.9
Middle Atlantic and East North Central States [3]	5.7	4.1	20.9	7.9	17.7	17.3	12.3	16.4	11.3	8.6	18.2
South Atlantic and East South Central States [4]	.7	.6	2.3	.8	1.3	2.0	1.8	2.3	1.3	1.0	1.0
East of the Mississippi River	6.9	5.1	25.2	9.3	20.0	20.4	15.0	19.9	13.8	10.5	20.1
West North Central and West South Central States [5]	7.2	5.7	17.4	14.1	41.9	52.5	45.6	22.3	12.6	8.8	51.5
Mountain States [2]	11.5	10.4	18.3	25.3	25.4	12.6	19.0	11.7	33.1	20.0	17.6
Pacific States	61.4	76.5	35.1	49.5	12.1	14.0	20.0	45.5	39.8	60.6	10.4
West of the Mississippi River	83.1	92.6	70.8	88.9	79.4	79.1	84.6	79.5	85.5	89.4	79.5
United States total	90.0	97.7	96.0	98.2	99.4	99.5	99.6	99.4	99.3	99.9	99.6
Foreign countries	10.0	2.3	4.0	1.8	.6	.5	.4	.6	.7	.1	.4
Grand total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] See note 1, table 6.
[2] Except Montana.
[3] Includes Delaware and Maryland.
[4] Does not include Delaware and Maryland.
[5] Includes Montana.

TABLE 8.—*Estimated number and percentage distribution of out-of-State passenger cars [1] in Michigan, 1930–31; Florida, 1933–34; and Arkansas, 1934–35, by area of origin*

Area of origin [2]	Michigan		Florida		Arkansas	
	Number of cars	Percentage of total	Number of cars	Percentage of total	Number of cars	Percentage of total
New England States	23,675	1.0	12,530	3.0	3,838	0.4
Middle Atlantic States	[3] 134,948	5.7	56,500	13.5	16,310	1.7
East North Central States	1,877,427	79.3	44,930	10.8	74,833	7.8
South Atlantic and East South Central States	[4] 56,820	2.4	[5] 274,450	65.7	230,256	24.0
East of the Mississippi River	2,092,870	88.4	388,410	93.0	325,237	33.9
West North Central and West South Central States	158,622	6.7	[6] 21,810	5.2	597,706	62.3
Mountain and Pacific States	49,718	2.1	5,610	1.3	34,538	3.6
West of the Mississippi River	208,340	8.8	27,420	6.5	632,244	65.9
United States total	2,301,210	97.2	115,830	99.5	957,481	99.8
Foreign countries	66,290	2.8	2,130	.5	1,919	.2
Grand total	2,367,500	100.0	417,960	100.0	959,400	100.0

[1] Does not include those driving through or making trips of less than 1 day.
[2] See note 1, table 6.
[3] Includes Delaware and Maryland.
[4] Does not include Delaware and Maryland.
[5] Does not include Mississippi.
[6] Includes Mississippi.

highway transportation of that State, shows that 79.3 percent originated in neighboring States—Ohio, Indiana, Illinois, and Wisconsin—5.7 percent in the Middle Atlantic States, including Delaware and Maryland; and 6.7 percent in the central plains States. In the aggregate, 88.4 percent of Michigan's total tourist traffic came from east of the Mississippi River, 8.8 percent from west of it, and 2.8 percent from Canada.

According to figures contained in the traffic survey, the estimated total number of out-of-State cars visiting Florida annually was 515,000, or an equivalent of 2.5 percent of all passenger vehicles registered outside of Florida in 1933. Florida drew 65.7 percent of its tourist traffic from the Southern States east of the Mississippi River except the State of Mississippi, which was combined in the report with the West South Central States; 27.3 percent was from the Northeastern States; and of the remainder 6.5 percent was from Mississippi and all States west of the Mississippi River, and 0.5 percent was from foreign countries.

CARS ON BUSINESS TRIPS CONSTITUTED LARGE PART OF OUT-OF-STATE TRAFFIC IN ARKANSAS

Although in the report of the traffic survey the total volume of out-of-State passenger-car traffic in Arkansas was estimated to be approximately 1,492,000 cars annually, a large part of these were making trips of less than 1 day's duration, many of these being recorded in the vicinity of Memphis, Tenn. In estimating the relative importance of recreational travel to this State, allowance should be made for business travel. This may be partly accomplished by deducting 532,600 cars reported as making trips of less than 1 day, leaving approximately 959,400 out-of-State cars annually visiting Arkansas for more than 1 day. An unknown portion of this traffic was for business purposes, since 60.8 percent of the total out-of-State traffic was classified as business. Probably the business traffic was much more heavily represented in the 532,600 cars visiting the State for less than 1 day than in the 959,400 cars staying longer, but such traffic undoubtedly constituted an important part of both groups.

Distributed according to point of origin, 62.3 percent of out-of-State passenger-car traffic in Arkansas originated in neighboring Central Plains States west of the Mississippi River; 33.9 percent came from east of the Mississippi, 24 percent being from the Southeastern States; 3.6 percent from the Mountain and Pacific States; and only 0.2 percent from foreign countries.

The data presented in table 8 pertain to out-of-State passenger vehicles remaining in the respective States more than 1 day. They are not an accurate measure of recreational travel but may be taken as an indication when studied in conjunction with other data collected in the surveys.

Twenty-five percent of the vehicles visiting Florida were on business trips and an additional 11 percent were there partly for business reasons. The average stay of vehicles visiting Florida was 29.4 days. Those visiting Michigan and Arkansas stayed, on the average, 11 days and 4.6 days, respectively. The percentage of business travel from outside the State was not determined for Michigan, and for Arkansas it was determined only for the total out-of-State travel, including visits of

less than 1 day. Of this total traffic 60.8 percent was for business reasons.

Figures from widely scattered parts of the country give evidence of the accuracy of the statement that the volume of tourist traffic tends to vary inversely with the distance between origin and destination. A count of highway traffic in Rhode Island was made in the summer of 1934. Of all cars of out-of-State registration, 75.5 percent were from other New England States, 19.4 percent from New York and Ohio, and only 5.1 percent from all other places.

Estimates by the New England Council show that about 80 percent of New England's recreational prospects are within the area east of the Mississippi and north of the Ohio River, including West Virginia, Maryland, and Delaware; and that New York City alone constitutes the primary market for tourist trips to New England.

A check of cars of out-of-State registration entering California during the first 8 months of 1935, made by

SEVERAL STATES INVITE AND ATTRACT OUT-OF-STATE TOURIST TRAFFIC BY PROVIDING FREE INFORMATION BOOTHS AND REST ROOMS ALONG THEIR HIGHWAYS. THIS STATION IS OPERATED BY THE TEXAS HIGHWAY DEPARTMENT TO DISSEMINATE INFORMATION ABOUT THE TEXAS CENTENNIAL EXPOSITION.

the department of motor vehicles, showed that 16.3 percent of these came from Washington and Oregon; 26.7 percent from the four nearby Mountain States, Arizona, Colorado, Utah, and Nevada; 10.5 percent from Texas and Oklahoma; 4.8 percent from Illinois; and 4 percent from New York. Thus, 62.3 percent of this traffic came from 10 States, 8 of which are comparatively near and directly accessible, while the other 2 are fertile fields of origin of tourist traffic.

An analysis of tourist traffic, made by the Texas State Highway Department in connection with the Texas Centennial in 1936, shows that the greatest number of tourist cars came from the neighboring States—Oklahoma, Louisiana, and New Mexico—with California in fourth place, followed by Missouri, Illinois, Tennessee, Mississippi, Indiana, New York, Michigan, and Wisconsin. In this case, also, it appears that the greatest volume of tourist traffic came from nearby States, supplemented in large measure by visitors from the Northeastern States.

DATA ON PURPOSE OF VISITS BY OUT-OF-STATE CARS DISCUSSED

When interpreted in a rather broad sense, "tourist" travel includes not only persons traveling for pleasure, health, or education, but also those traveling primarily for business. The business man from beyond the borders of a given locality utilizes the same accommodations and spends his money for much the same things as the tourist who is traveling solely for pleasure. On the other hand, a considerable part of out-of-State automobile travel is made up of cars making short trips which involve no overnight stops, and which in many cases contribute but a small part of the total tourist expenditures.

Figures that throw light on the purpose of travel are found in the proportion of pleasure and business travel reported by passenger-car owners in the States of New York and Minnesota. In New York, 55.8 percent of the owners questioned reported that they were traveling in their home State on business and 44.2 percent were traveling for pleasure. Similar figures for passenger car owners in Minnesota showed that 52.7 percent were traveling on business and 47.3 percent for pleasure.

Tourist data for Florida show that a high percentage of visits to that State were made for a combination of reasons. This may have resulted from the fact that the tourist questionnaire card used in Florida contained an enumeration of a number of purposes which were not mutually exclusive as the simple designations "business" or "pleasure" are. It was accordingly necessary to make an additional analysis of these mixed purposes. In the second column of data relating to Florida in table 9 are shown the additional percentages of visiting cars which reported each of the purposes in conjunction with one or more of the others listed. Thus the occupants of 28.4 percent of all out-of-State cars were visiting Florida solely for pleasure, and those of 25.6 percent were there exclusively on business. Of the 41 percent who were there for a combination of reasons, 11.3 percent of all out-of-State cars were traveling partly on business, 18.2 percent partly to visit friends, etc.

A much higher percentage of travel for pleasure is indicated by the figures for Wisconsin in 1931, when 73.2 percent of visiting automobiles were reported as being used for pleasure, and 26.8 percent for business or other reasons. The record of purpose of travel declared by operators of out-of-State passenger cars in Arkansas, including those making trips of less than 1 day, showed a definite tendency in the opposite direction, 60.8 percent of such trips being for business and 39.2 percent for pleasure.

TABLE 9.—*Purpose of visit of out-of-State passenger cars in Wisconsin, Florida, and Arkansas*

Purpose of visit	Percentage of out-of-State cars in—			
	Wisconsin	Florida		Arkansas [1]
		Exclusively	Partly	
Business	11.3	25.6	11.3	60.8
Pleasure:				
Vacation	28.8			
Visiting friends, etc.	19.7	12.7	18.2	
Scenery	16.5	10.2	25.4	
Fishing	5.2	3.6	18.0	
Sporting events		1.9	9.7	
Good roads	3.0			
Total pleasure	73.2	28.4		39.2
En route to other States	14.0	.9	12.4	
Other purposes	1.5	4.1	2.8	
Combinations		41.0		

[1] Includes cars making trips of 1 day or less.

Photo by Hileman

GLACIER NATIONAL PARK ANNUALLY ATTRACTS TOURISTS FROM FAR AND NEAR.

Only infrequently have published statements been found in which both the number of tourist automobiles and the actual or average number of passengers were given. An estimate by the American Automobile Association placed United States automobile tourist traffic during 1928 at 44,000,000 persons traveling in 11,000,000 cars, representing an average of 4 persons to a car. The estimate of that association for 1936 showed approximately the same number of cars carrying 37,000,000 passengers, or an average of 3.4 persons per car.

A combination of other figures for various States, covering different years from 1929 to 1935 and aggregating more than 35,000,000 tourists traveling in nearly 12,000,000 automobiles, gives an average of about 3 persons to a car; but if these figures are divided according to groups by years, an average of 3.1 passengers per car is shown for the earlier years, 1929 to 1931, inclusive, and an average of 2.8 passengers per car for the later years, 1932 to 1935.

TOURISTS MAKE LONGEST VISITS IN LOCALITIES WHERE AGREEABLE CLIMATE IS THE ATTRACTION

The average number of passengers per car visiting Michigan, Wisconsin, Florida, and Arkansas, and the average length of stay of automobile tourists in each State are shown in table 10, together with estimates of the total number of tourists and tourist-days based on

those averages. The Wisconsin figures for 1931 show the highest average number of passengers per car and the second highest average length of stay, making an estimated total of 102,646,500 tourist days spent in the State during that year. In point of total tourist days spent annually in each State at the indicated time of the survey, Michigan, Florida, and Arkansas were next in importance in the order named. Although a comparatively large number of individual tourists visited the State, a smaller number of tourist days were spent in Arkansas than in any of the other three States, because the average length of visit was exceptionally short.

The American Automobile Association's estimate for 1928 indicated the length of the average automobile tourist trip as approximately 15.5 days. This is significant, in view of the fact that two weeks is probably the average vacation period of the majority of people in the United States. The entire duration of such trips is not necessarily spent in a given area, or even within a single State, but the average for all tourists becomes important as a standard of reference for the comparison of data relating to local or regional tourist traffic.

TABLE 10.—*Estimated total annual automobile tourist traffic and length of stay in Michigan, Wisconsin, Florida, and Arkansas*

State and year	Estimated number of cars annually [1]	Estimated number of persons		Average length of stay per car in days	Estimated number of person-days spent in State annually
		Average per car	Total annually		
Michigan, 1930–31	2,367,500	2.82	6,676,350	11.0	73,439,850
Wisconsin, 1931	1,902,500	3.27	6,221,000	16.5	102,646,500
Florida, 1933–34	417,960	2.71	1,132,672	29.4	33,300,557
Arkansas, 1934–35	959,400	2.48	2,379,312	4.6	10,944,835

[1] Does not include those driving through or staying less than 1 day.

The average duration of visit in a designated area is a fairly accurate index of the type of attraction that draws the tourists. In those sections where either summer or winter climate is the inviting factor, the average length of stay tends to be longer than in others where the lure is of a scenic or historical nature. In one case the tourist comes to reside in the locality for a period, whereas in the other he travels only to see the place and then to pass on. The mild winter climates of both southern California and Florida make these two localities competitors for tourists who wish to escape cold weather in other parts of the country. The average length of visit to southern California in 1931 was 38.3 days, with tourists remaining about three times as long in winter as in summer. Visitors remained in Florida an average of 29.4 days in 1934–35. With the further extension of the Inter-American Highway, it is possible that Mexico and Central America will also enter this field of competition for winter-tourist traffic.

Visitors to northern and central California made an average stay of 11.5 days, but the average visit in the late spring, summer, and early fall, was five times as long as during the other 6 months of the year. In Colorado the average tourist visit was 14.8 days, with the length of stay during June, July, and August, averaging 20 days, and during the rest of the year 8 days. Visitors to Michigan and Wisconsin stayed an average of 11 and 16.5 days, respectively, these periods approximating those for northern and central California and Colorado. It would seem that the tourist appeal of these four States depends in part on climate and in part on scenic attraction.

In contrast with these, the average tourist visit of 4.6 days in Arkansas at the time of the traffic survey, and of 3 days in Arizona in 1936, indicates the transient nature of tourist traffic in these States. It was also reported that of the one and a quarter million tourists estimated to have visited Vermont in 1935, the majority were merely passing through the State.

OUT-OF-STATE CARS IN 11 WESTERN STATES TRAVELED 232 MILES DAILY

Another index of the more or less transient nature of tourist traffic is found in figures of average daily travel by out-of-State passenger cars in various localities. Although no figures are available in the survey of traffic in the 11 Western States regarding the length of stay of out-of-State passenger cars, the average daily travel by such cars in the respective States is shown. The average for such traffic in all 11 States was 232 miles per day, varying between a high of 247 miles a day in California and a low of 189 miles a day in Washington. It should be remembered, however, that the broad extent of land area and the greater distances between centers of population and between points of interest in the more sparsely populated Western States would account for a considerably higher average daily car mileage in those sections, in contrast with similar figures for cars traveling in the Eastern States.

Comparable basic figures and estimates of total annual travel by out-of-State passenger cars in Michigan, Wisconsin, and Florida, are shown in table 11. Although generalizations based on as few as three States are inadvisable, these figures show certain relationships that appear to be reasonable. The average daily travel by out-of-State cars was greatest in Michigan where the average length of stay was least, and least in Florida where the duration of visit was longest, suggesting that in those localities where tourist traffic is of a more permanent nature the average daily mileage of travel is less, and vice versa.

TABLE 11.—*Estimated total annual number of and travel by out-of-State passenger cars visiting Michigan, Wisconsin, and Florida*

State and year	Estimated number of cars annually [1]	Average number of days per car	Total number of car-days annually	Average travel		Estimated total travel annually
				Per day	Per trip	
				Miles	*Miles*	*Car-miles*
Michigan, 1930–31	2,367,500	11.0	26,042,500	74.2	816	1,932,353,500
Wisconsin, 1931	1,902,500	16.5	31,391,250	35.6	587	1,117,528,500
Florida, 1933–34	417,960	29.4	12,288,024	22.0	647	270,336,528

[1] Does not include those driving through or staying less than 1 day.

Not very many years ago, hotels and regular lodging houses were the only places where automobile tourists could find lodging. It has been estimated that approximately 73 percent of all automobile tourists in the United States stopped at hotels as recently as 1928 and 1929, the remaining 27 percent utilizing other types of accommodation. A similar estimate for New England in 1929 indicated that 70 percent of tourists visiting that area patronized hotels and 30 percent patronized all other types of accommodation.

But during recent years, both the number and variety of types of accommodation for automobile tourists have increased, along with the development of other facilities for their convenience. Private homes have been opened to tourists; tourist cabins and camp sites have

been established along the principal highways; and in those areas where there is a considerable amount of tourist traffic, cottages and apartments catering especially to tourists have been equipped. These developments have effected great changes in the distribution of tourist patronage of various types of accommodation.

The latest addition to the list of available accommodations for automobile tourists is the house trailer. The improvement and relatively widespread use of this newest feature of tourist travel have caused repercussions in various fields. Transient and semipermanent trailer camps have been established in many tourist areas, while such camps have been prohibited at some of the more exclusive resorts; and taxation experts have been busy with the problem of devising ways and means of assessing and collecting taxes on these rolling homes. Within the past year house trailers have increased amazingly in both number and variety, so that they may be expected to constitute an important factor in future studies of tourist traffic.

CLASSES OF ACCOMMODATION USED BY TOURISTS IN THREE STATES LISTED

Definite information regarding the type of accommodation used was obtained in connection with the Michigan, Florida, and Arkansas traffic surveys. The estimated number of passenger cars, tourists, and tourist-days for each of these States, classified according to type of accommodation, are shown in table 12. The distribution of the total number of tourist-days, by type of accommodation, is also shown graphically in figure 3.

The Michigan traffic survey shows that passengers of 31.1 percent of out-of-State cars making overnight stops in that State stayed at hotels; 10.1 percent patronized tourist camps; 35.4 percent were visiting friends; 19.1 percent had their own summer homes; while 4.3 percent used various other types of accommodation. Passengers of a considerable proportion of cars visiting Florida stopped at hotels, and a much smaller proportion were visiting friends or staying in their own homes. A great variety of accommodations for tourists was reported in Florida, where apartments and cottages, tourist homes, and tourist camps took care of the passengers of more than 20 percent of all tourist cars.

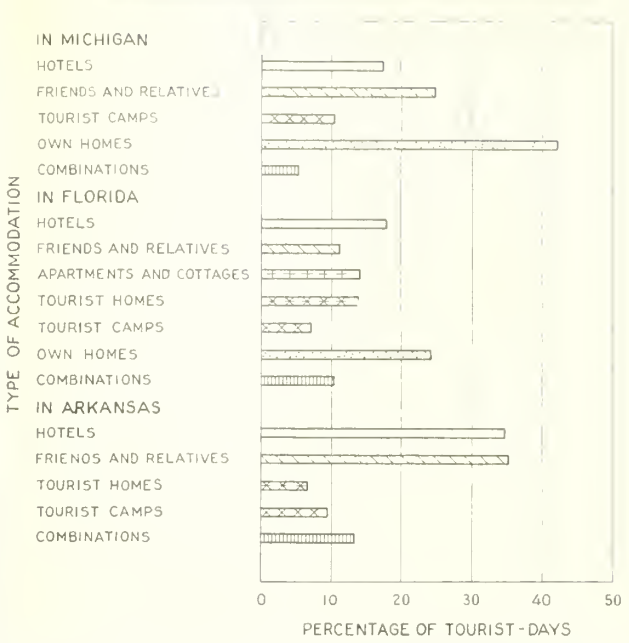


FIGURE 3.—PERCENTAGE DISTRIBUTION OF NUMBER OF TOURIST DAYS SPENT IN MICHIGAN, FLORIDA, AND ARKANSAS, BY TYPE OF ACCOMMODATION.

TABLE 12.—*Estimated annual number of out-of-State passenger cars, number of tourists, and duration of stay in Michigan, Florida, and Arkansas, by type of accommodation* [1]

Type of accommodation	Passenger cars		Tourists			Duration of stay		
	Number	Percentage of total	Persons per car	Number	Percentage of total	Days per person	Number of person-days	Percentage of total
IN MICHIGAN								
Hotels	735,000	31.1	2.27	1,668,450	25.0	8.2	13,681,290	17.4
Friends and relatives	837,500	35.4	3.10	2,596,250	38.8	7.5	19,471,875	24.8
Apartments and cottages								
Tourist homes								
Tourist camps	240,000	10.1	3.17	760,800	11.4	10.6	8,064,480	10.3
Own homes	452,500	19.1	3.05	1,380,125	20.6	24.0	33,123,000	42.2
Combinations	102,500	4.3	2.74	280,850	4.2	14.7	4,128,495	5.3
All types	2,367,500	100.0	2.82	6,686,475	100.0	11.7	78,469,140	100.0
IN FLORIDA								
Hotels	162,590	38.9	2.28	370,705	32.7	19.5	7,228,748	18.1
Friends and relatives	87,770	21.0	3.09	271,209	23.9	16.1	4,366,465	10.9
Apartments and cottages	22,150	5.3	2.95	65,342	5.8	87.7	5,730,493	14.4
Tourist homes	41,380	9.9	2.83	117,105	10.3	48.2	5,644,461	14.2
Tourist camps	26,330	6.3	3.28	86,362	7.6	33.5	2,893,127	7.3
Own homes	32,180	7.7	2.84	91,391	8.1	106.3	9,714,863	24.4
Combinations	45,560	10.9	2.87	130,757	11.6	32.6	4,262,678	10.7
All types	417,960	100.0	2.71	1,132,871	100.0	25.2	39,840,835	100.0
IN ARKANSAS								
Hotels	474,500	49.5	1.77	839,865	35.9	6.2	5,207,163	34.8
Friends and relatives	264,100	27.5	3.17	837,197	35.8	6.3	5,274,341	35.2
Apartments and cottages								
Tourist homes	32,800	3.4	2.84	93,152	4.0	11.0	1,024,672	6.8
Tourist camps	113,400	11.8	3.20	362,880	15.5	3.9	1,415,232	9.5
Own homes								
Combinations	74,600	7.8	2.74	204,404	8.8	10.0	2,044,040	13.7
All types	959,400	100.0	2.44	2,337,498	100.0	6.4	14,965,448	100.0

[1] Not including those driving through or staying less than 1 day.

In Arkansas the passengers of 49.5 percent of out-of-State cars were reported as stopping at hotels; 27.5 percent stayed with friends; 11.8 percent stopped in tourist camps; and 3.4 percent patronized tourist homes. The high percentage of hotel patronage in this State gives further emphasis to the fact, already pointed out, that an exceptionally large proportion of out-of-State cars were in Arkansas on business.

In each of these three States cars patronizing hotels had the smallest average number of passengers, while cars most heavily laden with passengers were visiting friends or stopping at tourist camps. The average number of passengers reported for cars stopping at tourist homes in both Florida and Arkansas was approximately the same.

Twenty-five percent of the individual automobile tourists patronized hotels in Michigan, 32.7 percent in

Camp Trailers Like This One Were the Forerunners of the House Trailers, Whose Numbers Have Increased Rapidly During the Past Several Years.

Florida, and 35.9 percent in Arkansas. In Michigan nearly 6 out of 10 visitors stayed with friends or in their own homes, but in Florida only about 1 out of 3 was so provided for. Tourist homes were more popular than tourist camps in Florida, while in Arkansas the reverse was true.

Visitors who lived in their own summer or winter homes in both Michigan and Florida, stayed a much longer time than tourists using other types of accommodation. Those who occupied rented apartments or cottages in Florida also made comparatively long visits. The average length of stay in hotels or with friends in both Michigan and Arkansas was about 1 week, while in Florida an average stay of between 2 and 3 weeks was reported for each of these groups.

DISTRIBUTION OF TOURIST EXPENDITURES DISCUSSED

The patronage accorded each type of accommodation in these three States is shown in the last two columns of table 12. In Michigan, 67 percent of the total number of tourist days was spent in the visitors' own homes or with friends, and furnished no direct business to those providing transient accommodations for tourists. Nevertheless, Michigan hotels received more than 13,681,000 person-days of patronage by automobile tourists, representing a larger volume of such business than the combined total for both Florida and Arkansas. In Florida, 35 percent of all tourist days was spent in the visitors' own homes or with friends and relations, and in Arkansas, which is not important as a summer home section, 35 percent of the total number of tourist days was spent with friends and relations. In Michigan and Florida, 17 and 18 percent, respectively, of all tourist days were spent in hotels; but in Arkansas, where a large percentage of out-of-State cars making business trips was recorded, hotels received 35 percent of all such patronage. The many different kinds of accommodation offered the tourist in Florida probably accounts in large measure for the more even distribution of actual tourist time among them.

Estimates of the distribution of American tourist expenditures throughout the United States have been made from time to time. In one of these estimates the allocation was as follows: Food, 21 percent; lodging, 20 percent; transportation, 20 percent; retail stores, 25 percent; recreation and amusement, 8 percent; and miscellaneous items, 6 percent. This estimate relates to expenditures by all classes of tourists, regardless of the mode of transportation. Nevertheless, it seems likely that, for the country as a whole, approximately the same distribution of expenditures by automobile tourists would be found, except that comparatively lower transportation cost might result in apparently higher percentages for other items.

The distribution of expenditures in individual States, however, may be expected to show considerable variation from the general or national average. The distribution listed in the preceding paragraph was endorsed by the New England Council as being representative of expenditures by all classes of tourists in the New England States. But an estimate by Californians, Inc., showed a much higher proportion spent for food and lodging by all classes of tourists in northern and central California in 1935 with relatively smaller amounts spent for clothing and general merchandise. The percentages in this distribution were: Food, 36.9 percent; lodging, 28.2 percent; gas, oil, and car expenses, 10.1 percent; local transportation, 5.3 percent; clothing, 5.5 percent; personal expenditures, 4.7 percent; recreation, 3.7 percent; souvenirs, 3 percent; and camera supplies, 2.6 percent.

The foregoing estimates have been based on the consideration of what the tourist received for his money. Another type of distribution takes into account the dissemination of tourist expenditures among the various business enterprises of the community in which the money was spent. Such an analysis of 1930 tourist expenditures in Maine was made by the Maine Development Commission, with the following resultant distribution: Hotels and sporting camps, 16 percent; rooms, overnight camps, and eating places, 7 percent; boys' and girls' camps, 5 percent; groceries, 11 percent; all other stores, 10 percent; garages and filling stations, 9 percent; construction work, 7 percent; amusements and sports, 6 percent; utilities and transportation, 4 percent; insurance, 3 percent; farm produce and fuel, 3 percent; direct employment, 2 percent; antiques and gifts, 2 percent; and all other items, 15 percent.

A more recent estimate of the distribution among various classes of business of approximately 5 billion dollars spent by the American tourist in the United States in 1936 was made by Roger Babson, a nationally recognized statistician. This estimate shows that retail merchants received 25 percent; restaurants and cafes, 21 percent; hotels and camps, 17 percent; gasoline stations, 12 percent; theaters and amusements, 9 percent; transportation (rail, bus, etc.), 7 percent; confectionery stores, 5 percent; and other kinds of business, 4 percent.

TYPE OF ACCOMMODATION PATRONIZED BY TOURISTS AFFECTED DISTRIBUTION OF EXPENDITURES FOR FOOD, LODGING, TRAVEL, ETC.

Estimates of total expenditures by automobile tourists were made in connection with the highway traffic surveys in Michigan and Florida. These figures are presented in table 13, distributed according to type of accommodation patronized. In each State the expenditures made by persons stopping at hotels were approximately the same as the expenditures by those living in their own summer or winter homes, but in Michigan each of these represented a greater part of the total than in Florida.

The percentage of total expenditures made by tourists visiting friends and relatives was also much greater in Michigan than in Florida, because both the number and the average daily expenditure by this class of tourists were greater in Michigan. On the other hand, it should be observed that a greater variety of accommodations for tourists was listed in Florida, where tourists patronizing furnished apartments and cottages, tourist homes, tourist camps, and various combinations, made almost one-third of the total tourist expenditures, in contrast with about 15 percent made by those using tourist camps and miscellaneous accommodations in Michigan. In both States, tourists making no overnight stops were credited with only 0.4 percent of the total tourist expenditures.

TABLE 13.—*Distribution of estimated total annual expenditures by automobile tourists in Michigan and Florida, by type of accommodation*

Type of accommodation	Estimated annual expenditures			
	In Michigan [1]		In Florida [2]	
	Amount	Percentage of total	Amount	Percentage of total
Hotels	$98,400,000	35.9	$27,932,000	30.9
Friends and relatives	33,700,000	12.3	5,257,000	5.8
Apartments and cottages			9,033,000	10.0
Tourist homes			8,896,000	9.8
Tourist camps	25,600,000	9.3	2,320,000	2.6
Own homes	99,000,000	36.1	27,947,000	30.9
Combinations	16,300,000	6.0	8,704,000	9.6
Through traffic	1,100,000	.4	369,000	.4
All types	274,100,000	100.0	90,458,000	100.0

[1] For 1930-31.
[2] For 1933-34.

On the basis of unit cost of designated items of expenditure by tourists patronizing various types of accommodation, the distribution of expenditures of each class, as shown in table 14, has been derived. This distribution is also shown graphically in figure 4. For all tourists the proportional cost of food and lodging was higher in Florida, and the percentages for car operation and miscellaneous expenditures were relatively greater in Michigan. Detailed analysis would reveal many causes of this difference in distribution of expenditures in these two States, but the most obvious are probably the greater average length of stay in Florida, and the greater average daily car mileage in Michigan.

In each State the highest relative cost of lodging was paid by those in their own homes; the estimated daily cost of this item in Michigan was based on an average rental value of summer homes during the 100-day tourist season, and in Florida on an average annual

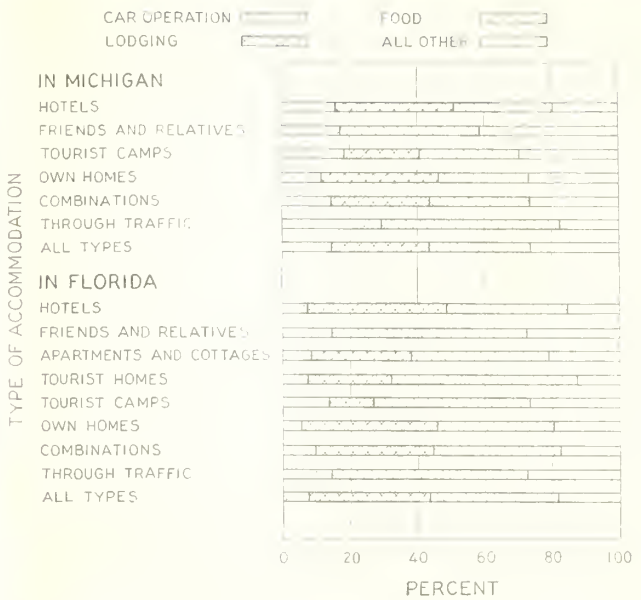


FIGURE 4.—PERCENTAGE DISTRIBUTION OF ESTIMATED ANNUAL EXPENDITURES BY AUTOMOBILE TOURISTS IN MICHIGAN AND FLORIDA, BY TYPE OF ACCOMMODATION.

cost of operating and carrying charges distributed over a season of 160 days. In both States the relative cost of hotel accommodations was only slightly lower than that of operating private homes, and the lowest proportional cost of lodging was paid in tourist camps. Because no allowance was made for cost of lodging for groups of tourists visiting friends, or making no overnight stops in the State, the relative part of total expenditures spent for food was greatest for these two classes.

TABLE 14.—*Distribution of estimated expenditures by automobile tourists patronizing various types of accommodation in Michigan and Florida*

Type of accommodation	Percentage of estimated expenditures for—				
	Car operation	Personal expenses			
		Lodging	Food	All other	Total
IN MICHIGAN					
Hotels	15.7	35.3	29.4	19.6	84.3
Friends and relatives	17.6		41.2	41.2	82.4
Apartments and cottages					
Tourist homes					
Tourist camps	18.4	22.2	29.7	29.7	81.6
Own homes	11.3	35.5	26.6	26.6	88.7
Combinations	14.5	29.4	29.9	26.2	85.5
Through traffic	29.3		53.0	17.7	70.7
All types	14.6	29.3	30.0	26.1	85.4
IN FLORIDA					
Hotels	7.3	41.5	36.1	15.1	92.7
Friends and relatives	14.5		57.8	27.7	85.5
Apartments and cottages	8.6	29.8	40.8	20.8	91.4
Tourist homes	7.2	25.1	35.1	12.6	92.8
Tourist camps	13.7	13.3	46.4	26.6	86.3
Own homes	5.1	45.9	29.3	19.7	94.9
Combinations	9.2	35.7	37.3	17.8	90.8
Through traffic	14.2		58.7	27.1	85.8
All types	7.5	36.2	38.1	18.2	92.5

From these figures it may be seen that the type of accommodation patronized by tourists has considerable effect upon the relative distribution of expenditures made by each class. On the other hand, the reasonable degree of comparability found between the distribution

Adequate Parking Areas, Such as This One at Mount Vernon, Are a Necessary Accommodation for Motor Tourists at Historical and Scenic Attractions.

of expenditures by automobile tourists in Michigan and Florida, and by all classes of tourists in northern and central California, previously cited, bears out the assumption that the nature of expenditures made by tourists is only slightly affected by the type of transportation used.

Although available information relating to tourist travel by automobile within the United States is, as yet, fragmentary and so dissimilar as to make accurate comparison or summary impossible, the general review of such information may suggest a standard by which greater accuracy and uniformity can be obtained in the future. Knowledge of the actual number of and mileage traveled by out-of-State automobiles to be expected in a given area at a certain time is of value to officials who plan local programs of highway development and maintenance. Facts regarding the origin and distribution of patronage of automobile tourists visiting their districts are of interest to organizations and business enterprises, especially to those that cater directly to tourist trade. From the point of view of the tourist himself, analysis of the nature of accommodations for travelers, the daily total and per-capita costs of various items, and the distribution of expenditures on the basis of the type of accommodation used, are of particular interest.

If it were possible to obtain, from a large number of automobile tourists visiting various points of interest throughout the country, exact information regarding each of the points touched upon in the preceding discussion, the results would undoubtedly be of great value from both a local and a National point of view.

BIBLIOGRAPHY ON HIGHWAY LIGHTING AVAILABLE

A bibliography on highway lighting, covering primarily the years 1913 to 1936 and accenting more recent developments, has just been published by the Bureau of Public Roads of the United States Department of Agriculture as Miscellaneous Publication No. 279.

The references are arranged according to the time of publication under each of these years, with a collected author index at the end. Articles in French, German, Spanish, and other languages, as well as in English, are listed.

Highway engineers and city officials will find this bibliography a guide to literature on the latest developments in lighting streets and highways, bridges, viaducts, tunnels, and causeways. For example, material is available on new types of lamps, and on the photoelectric cell which switches the system on and off, depending upon the natural light available, as well as on the French system of equalizing tunnel illumination with the outer light at all times.

This annotated bibliography may be obtained free from the United States Department of Agriculture, Washington, D. C.

INDEX TO PUBLIC ROADS, VOLUME 17, TO BE AVAILABLE SOON

The index to volume 17 of Public Roads is now being printed and will be available soon. In addition to the index, a chronological list of articles and a list of authors are given. The index will be sent free to subscribers to Public Roads requesting it. Requests should be addressed to the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C.

EFFECT OF HIGHWAY DESIGN ON VEHICLE SPEED AND FUEL CONSUMPTION STUDIED IN OREGON

A publication reporting experiments performed to measure the effect of highway design on vehicle speed and fuel consumption has recently been issued by the Oregon State Highway Commission as Highway Department Technical Bulletin No. 5. The report was prepared under the supervision of Mr. John Beakey, Traffic Engineer.

The primary purpose of the investigation was to determine the effect of grades on fuel requirements. However, before actual tests could be made it was found neeessary to broaden the scope of the work to include tests on eurvature and surfaee types in order to eliminate, as much as possible, those variables.

Since motor fuel eonsumption makes up a large part of the total vehicle operating eost, the importance of better understanding faetors affecting this variable is apparent. Among these faetors, speed and gradient are by far the most important.

The report presents the results of tests made over a period of a year and a half on passenger cars and heavier equipment operating under both controlled and actual operating eonditions. Only a limited number of vehicles were tested, but the results serve to point the way toward a more accurate analysis of the effect of highway design on vehieular operating eosts.

The eonelusions reached in this study are as follows:

A. Relative to Level Road-Grade Equivalents.

1. The potential energy theory heretofore applied to grade reduetion problems fails to furnish a true method for the determination of level road equivalents sinee in that method no consideration is given to the dissipation of stored energy when descending grades.

2. The true measure of level road rise equivalents must take into eonsideration both up and down grade operation, and should be based upon total operating eosts rather than upon fuel eosts alone. When these faetors are duly eonsidered, the results of these tests indieate the following general relationships for motor-vehiele equipment in eurrent use.

(*a*) For modern passenger cars the level road equivalent of 1 foot of rise varies from 2.28 feet to a value less than 1 foot and is therefore, in general, negligible in grade reduction problems.

(*b*) For heavier equipment the level road equivalents are considerably larger, and, in general, increase with the percent of grade largely because of the necessity for shifting gears on grades, a necessity which does not exist in the case of passenger cars for the grades investigated. As an example, with a truck weighing 45,000 pounds gross the level road equivalents determined by these tests were as follows:

Percent of grade:	*Level road equivalent of 1 foot of rise (feet)*
1 percent	2. 20
2 percent	4. 45
3 percent	6. 65
4 percent	8. 90
5 percent	12. 00
6 percent	15. 20

(*c*) In general, the level road rise equivalents for automotive vehicles in any weight class can be determined from the formulas and curves given in the body of this report, once the fuel consumption on grades and on the level are known. These fuel consumption values, for heavy equipment, may be estimated very closely from the following formulas which are based upon the results of this investigation:

For level grade $C = 0.0001283W^{0.712}$
For 1 percent grade $C = 0.0001179W^{0.723}$
For 2 percent grade $C = 0.0000954W^{0.750}$
For 3 percent grade $C = 0.0000731W^{0.785}$
For 4 percent grade $C = 0.0000542W^{0.825}$
For 5 percent grade $C = 0.0000373W^{0.876}$
For 6 percent grade $C = 0.0000260W^{0.928}$

Wherein "C" is the consumption of fuel in gallons per mile, and "W" is the gross weight of the vehicle in pounds.

3. The above level road equivalents take into consideration both ascending and deseending grade movements. In those rather unusual eases where it becomes neeessary to eonsider the level road equivalent of 1 foot of rise for ascending grade movement only, the tables and formulas given in the body of this report furnish a basis for the determination of such equivalents.

B. Relative to Fuel Consumption (Light Vehicles).

4. For the average modern passenger car, fuel consumption at constant speed on ascending grades up to 6 pereent increases at a uniform rate with eaeh percent inerease in grade.

5. For the average modern passenger ear, fuel consumption at eonstant speed on descending grades up to 6 percent is, at the lower speeds, a time function depending upon the idling adjustment of the given vehicle. At all speeds at which throttle opening is required, fuel consumption decreases at a nearly uniform rate with each pereent increase in descending grade.

6. For the average passenger car, fuel consumption at eonstant speed for composite grades (i. e., both ascending and descending) increases with each percent increase in grade, the rate of inerease being somewhat greater for the steeper grades. The increase in fuel consumption for this class of vehicle, however, is generally so small as to be negligible unless traffic is abnormally dense, and for this reason grade reduetions below 6 percent ean generally be justified only when there is or will be a considerable volume of heavy truck traffic.

C. Relative to Fuel Consumption (Heavy Vehicles).

7. For heavy automotive equipment, fuel eonsumption is definitely a funetion of the percent or rate of grade beeause of the characteristie speed and gear employed on each grade.

8. Descending grade fuel consumption for heavy vehicles varies between wide limits due to the effect on speed of grade, length, curvature and weather conditions.

9. In general, fuel eonsumption for heavy equipment inereases with each percent increase in grade; however, no material saving is possible through the reduction of grades of 2 percent or less. This is true of gasoline powered trueks, and results from a limited number of Diesel powered trucks indicate that savings from grade reduetions will be proportionally the same.

10. In general, fuel requirements on a section comprising several different grades will be the same (except for the effect of vertieal curves, which is small) as that on the same length of constant grade of the same average rate, provided that eonditions of constant speed and uniform fuel mixture are maintained.

11. The time savings obtained from grade reduction in the case of grades up to 6 percent is of no material importance to light passenger vehicles but does affect truck operation on grades steeper than 2 percent. The relation between speed and gross weight for the six heavy vehicles included in these tests operating on grades may be expressed as follows:

For ascending grades:

Speed (m. p. h.)=60−0.5 W−4.33G.

For composite grades (ascending and descending):

Speed (m. p. h.)=60−0.5W−1.5G.

Where

W=The gross weight of the vehicle in thousands of pounds.

G=The percent of grade.

D. Relative to Road Design.

12. Power requirements and consequent fuel consumption for light vehicles will not be materially affected by road curvature of 6° or less if such curvature is properly superelevated. No tests were made on spiral curves such as are now standard for trunk highways in Oregon.

13. The difference in efficiency between a concrete road surface and a modern bituminous type is very slight when considered from a fuel consumption standpoint.

14. The above conclusions have dealt with conditions wherein the effective rise and fall has been decreased. Grade reductions in which the effective rise and fall is not decreased will result in no material savings in fuel consumption for light passenger cars, but will effect some saving in the case of heavy automotive units.

E. Relative to Diesel Powered Trucks.

15. Results from a survey comprising 100 vehicles in actual service show Diesel fuel consumption, expressed in gallons per mile, to be 40 percent less than gasoline in relatively level country and 45 percent less in mountainous country.

16. Reduction of those grades that will result in savings of fuel on both heavy and light equipment will yield greater fuel savings, on a ton-mile basis, with heavy than with light equipment. However, the resulting savings in cost of operation per ton-mile may be less on Diesel powered heavy equipment than on passenger cars due to the lower cost of fuel.

F. Relative to Automotive Equipment in General.

17. Passenger car operating costs are materially affected by carburetor and timing adjustments.

18. Fuel requirements even for a limited number of vehicles will show a wide variation depending upon individual characteristics.

19. Gasoline consumption will generally vary directly as power output over a considerable range, but air-fuel ratio may materially affect the linearity.

20. Any drop in air-fuel ratio, particularly noticeable at high and at low power requirements at constant speed, materially increases fuel consumption.

21. The exhaust gas analyzer used in these tests proved indispensable for duplication of results and for confirmation on the accuracy of the results of gasoline consumption tests on light vehicles.

22. The overall thermal efficiency of the average passenger car increased with an increase in engine load produced, either by an increase in speed or by operation on steep grades, or both. The peak efficiency was attained at a relatively high speed on a steep grade. Engine characteristics may cause the efficiency to drop when the engine is overloaded by speed and grade.

23. Heavy motor vehicle operating characteristics on grades vary considerably, depending on the engine type, characteristics, and motive power per ton of gross vehicle weight.

24. Heavy motor vehicles operating at practically constant engine speed have definite characteristic road speeds depending on the percent of the grade and the power per ton.

25. It is believed that the results of tests on heavy equipment conducted under actual operating conditions and modified by the methods of operation give more representative information than tests conducted at constant speed in each gear.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JULY 31, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 1,354,601	$ 677,300	58.5	$ 4,029,960	$ 2,014,975	177.3	$ 5,154,905
Arizona	$ 6,200	$ 4,260		1,616,256	1,074,250	51.7	552,620	337,105	20.4	1,868,924
Arkansas	146,390	146,390	7.3	3,629,617	3,624,220	219.7	305,722	305,081	32.4	2,322,961
California	2,255,703	1,293,618	61.5	6,473,132	3,418,794	94.2	2,057,326	1,115,649	49.4	3,044,825
Colorado	638,143	356,958	19.3	2,874,538	1,597,574	102.0	124,133	64,552	1.5	2,344,136
Connecticut				745,158	370,176	8.7	277,140	130,933	1.4	1,504,408
Delaware				415,500	207,664	12.3	340,717	168,337	22.0	1,107,687
Florida	134,650	67,320	6.6	2,328,572	1,164,285	57.3	315,470	157,735	3.6	3,021,560
Georgia	490,777	245,388	26.9	3,435,437	1,717,703	175.8	2,068,335	1,034,167	71.8	5,665,575
Idaho	70,292	41,895	3.6	2,021,905	1,208,529	134.4	548,512	326,981	36.3	1,388,441
Illinois	772,315	351,683	28.5	10,437,547	5,203,482	306.5	4,597,943	2,296,110	106.4	3,018,562
Indiana	812,708	406,299	21.4	5,852,418	2,926,204	155.9	1,455,682	727,841	27.4	2,281,486
Iowa				6,236,724	2,780,672	187.3	2,614,190	1,235,130	76.0	2,078,549
Kansas	734,294	340,700	34.0	5,527,787	2,748,734	220.9	1,903,220	951,603	107.7	3,432,488
Kentucky	435,792	217,896	10.8	3,178,401	1,539,201	74.6	2,116,630	1,058,425	110.8	2,842,041
Louisiana	30,438	15,219	1.3	9,430,396	1,291,274	26.2	946,050	401,952	39.5	2,469,714
Maine	236,160	118,080	7.0	1,678,356	839,178	47.5	1,453,005	726,502	32.1	594,958
Maryland	180,456	90,228	2.1	1,439,650	719,792	21.7	497,318	248,659	7.4	2,036,129
Massachusetts				4,434,456	2,217,228	20.3	1,049,224	524,611	4.5	2,346,493
Michigan	1,842,300	921,150	59.7	5,961,012	2,980,506	138.0	4,067,251	1,955,900	80.9	871,605
Minnesota	752,420	376,210	41.1	3,303,003	1,583,382	146.2	2,685,158	1,342,579	75.0	2,715,136
Mississippi	129,800	64,900	4.1	3,631,910	1,815,900	184.9	2,365,780	1,182,130	100.4	3,570,094
Missouri	1,814,658	904,948	52.9	7,119,277	3,388,843	339.0	3,781,232	1,587,033	170.2	2,892,528
Montana	1,297,870	727,532	90.1	2,915,147	1,635,609	196.6	655,180	306,584	31.1	2,759,860
Nebraska	444,831	222,416	56.9	5,159,123	2,598,924	524.3	1,351,354	644,659	91.7	2,651,767
Nevada	277,100	239,885	14.3	2,297,539	1,979,919	103.6	61,474	53,269		1,096,321
New Hampshire				568,857	281,163	7.8	33,344	15,949	.3	1,135,399
New Jersey				2,055,959	950,684	20.3	832,890	253,805	1.9	2,513,349
New Mexico	282,000	171,714	11.1	2,298,885	1,511,928	142.6	1,671,325	1,019,086	106.8	759,819
New York	1,104,224	543,801	15.5	19,338,073	9,018,916	321.2	3,258,250	1,603,955	48.6	2,804,550
North Carolina	687,011	343,505	40.5	5,092,734	2,420,237	326.0	1,225,711	562,705	29.6	3,723,913
North Dakota	93,310	93,310	17.7	1,193,800	1,173,290	212.1	511,762	511,762	70.5	3,943,871
Ohio	583,095	291,347	10.6	8,200,197	4,000,785	88.7	2,055,861	1,027,930	25.7	6,742,798
Oklahoma	497,608	261,487	18.3	3,331,226	1,727,836	112.5	1,672,447	877,955	95.7	3,803,069
Oregon	672,742	405,039	28.4	3,883,589	2,283,628	131.4	584,688	336,801	27.7	1,229,285
Pennsylvania	789,470	394,679	11.4	11,482,086	5,729,076	159.8	3,162,508	1,571,653	49.3	5,902,043
Rhode Island	59,100	29,550	.8	1,444,736	722,368	17.4	102,270	49,824	1.2	902,980
South Carolina	680,995	289,200	36.7	4,942,516	2,049,458	298.0	814,383	324,235	50.6	2,193,732
South Dakota	142,012	89,192	14.8	1,821,814	1,011,848	181.6	1,228,818	679,770	116.9	3,601,680
Tennessee	451,598	225,799	19.6	1,705,284	852,642	54.5	365,580	182,790	14.6	5,254,798
Texas	2,211,809	1,105,365	178.0	11,597,498	5,778,831	688.0	1,340,359	669,893	59.9	8,747,209
Utah	101,208	72,462	6.2	1,132,450	804,656	107.0	252,670	176,290	25.4	1,533,249
Vermont	235,200	117,600	6.0	1,110,679	498,414	31.2	666,944	278,402	14.2	312,273
Virginia	360,120	180,060	17.7	3,220,072	1,551,838	102.7	399,501	199,751	17.9	2,911,523
Washington	604,380	317,700	46.4	2,002,151	1,048,971	32.3	1,264,488	661,598	26.1	1,397,037
West Virginia	330,204	165,102	11.6	1,399,472	699,384	33.9	717,264	354,227	21.6	2,457,878
Wisconsin	1,083,758	531,321	30.9	7,352,782	3,367,615	213.8	2,168,201	995,400	71.1	1,968,486
Wyoming	255,537	157,378	29.9	2,624,746	1,603,884	295.8	414,560	255,800	31.4	534,358
District of Columbia										1,227,500
Hawaii	236,496	117,895	4.4	616,749	301,733	12.8	371,350	182,080	5.4	625,000
Puerto Rico										
TOTALS	24,965,174	13,056,481	1,105.9	201,913,817	100,698,528	7,199.5	67,335,800	33,690,163	2,389.6	130,406,952

U. S. GOVERNMENT PRINTING OFFICE 3507

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JULY 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,364,997	$ 3,329,855	111.5	$ 690,450	$ 690,450	26.8	$ 80,572	$ 80,572	6.5	$ 50,238
Arizona	2,569,841	3,012,884	2,420,476	188.6	144,128	73,622	7.3				75,743
Arkansas	3,352,061	2,928,504	2,909,704	323.5	401,979	400,462	36.8				41,894
California	7,747,928	6,588,553	6,381,990	242.8	1,544,552	1,343,580	21.2				22,357
Colorado	3,395,263	2,345,433	2,279,520	101.0	89,598	89,596	6.0	8,200	8,200		1,017,947
Connecticut	1,418,709	639,975	589,124	4.7	542,320	525,090	15.7	318,290	253,845	2.3	50,650
Delaware	900,310	605,917	580,185	48.9	274,312	274,312	17.9	3,705	3,705		42,108
Florida	2,597,144	2,283,306	2,249,480	83.4	307,321	307,321	15.7				40,343
Georgia	4,988,967	995,153	976,998	61.5	1,418,207	1,418,207	82.1	1,642,117	1,642,117	76.0	951,646
Idaho	2,222,747	2,245,449	2,151,909	185.6	48,669	48,669	.3				22,169
Illinois	8,694,009	7,872,669	7,712,432	445.1	852,836	852,836	31.7	57,500	57,500	11.6	71,241
Indiana	4,941,255	3,879,534	3,661,055	158.2	1,331,677	1,273,455	80.2				6,745
Iowa	4,991,664	4,564,346	4,269,678	497.7	660,193	651,360	30.1	71,113	68,670	.7	1,955
Kansas	4,994,975	4,341,256	4,301,056	347.4	666,873	624,701	28.8	41,632	41,632	14.0	27,586
Kentucky	3,726,271	3,213,645	3,086,924	343.9	503,380	503,380	12.3	73,774	73,774	1.8	62,193
Louisiana	2,890,429	1,997,149	1,778,146	130.0	1,099,383	1,005,670	37.7	112,699	74,701	10.4	31,912
Maine	1,676,799	1,409,956	1,395,668	66.3	252,617	252,617	8.8	28,514	28,514	.7	
Maryland	1,750,738	475,773	469,405	17.6	747,571	747,571	17.4	193,134	159,302	6.4	374,460
Massachusetts	3,262,885	391,467	391,467	4.0	2,609,120	2,218,350	14.4	1,149,906	581,543	.8	71,525
Michigan	6,301,414	6,471,719	5,940,974	287.2	297,637	296,521	4.8	50,200	39,582	.3	24,337
Minnesota	5,277,145	5,764,522	4,821,900	869.2	676,607	431,309	33.3	42,575	23,936	.8	
Mississippi	3,457,552	2,467,736	2,463,917	176.1	897,215	896,175	58.6	39,400	39,400	1.5	58,060
Missouri	6,012,652	4,875,442	4,812,618	766.7	1,226,537	1,061,495	11.0	34,390	32,294		106,245
Montana	3,676,416	3,432,472	3,421,362	195.3	278,930	237,591	9.9	8,462	8,462		9,002
Nebraska	3,870,739	3,119,172	3,021,130	329.6	616,026	616,023	39.7	221,856	221,856	3.2	11,731
Nevada	2,243,074	2,232,578	2,168,521	109.9	86,696	71,696					2,857
New Hampshire	945,225	783,152	752,900	34.4	112,951	112,166	3.3	53,189	53,189	2.5	26,970
New Jersey	3,129,805	1,063,016	1,060,016	16.8	1,992,109	1,978,954	17.3	63,390	63,390	1.9	27,445
New Mexico	2,871,397	2,348,744	2,343,986	182.9	406,063	406,063	20.8	63,139	63,139	8.4	58,209
New York	11,046,377	10,292,702	9,833,640	157.6	669,044	669,044	12.6	52,478	52,478		491,215
North Carolina	4,720,173	3,307,162	3,274,392	220.7	1,434,870	1,397,070	70.0	39,700	39,700	.5	9,011
North Dakota	2,867,245	2,309,727	2,285,752	346.1	237,825	237,769	17.3	329,291	329,291	52.7	14,433
Ohio	7,670,815	4,074,665	4,002,492	177.6	3,448,677	3,408,242	112.5	232,940	224,420	6.6	35,661
Oklahoma	4,580,670	4,055,057	3,962,854	385.6	471,156	471,096	15.0	189,950	129,057	8.5	17,662
Oregon	3,038,642	2,147,260	2,126,018	155.6	1,129,226	900,009	8.9				12,615
Pennsylvania	9,347,797	2,481,275	2,374,057	127.2	5,738,920	5,358,959	120.9	1,741,425	1,343,133	37.7	271,648
Rhode Island	989,208	1,109,360	986,896	18.8	2,240	2,240					72
South Carolina	2,702,012	1,949,844	1,843,768	213.2	870,510	803,007	36.3	21,242	21,242	1.5	33,995
South Dakota	2,976,454	2,190,419	2,187,340	410.5	748,857	748,857	82.3	24,860	24,860	2.8	15,397
Tennessee	4,192,460	2,646,689	2,619,897	105.1	1,294,797	1,294,797	42.1	126,650	126,650	1.4	151,117
Texas	11,989,350	12,437,179	11,416,518	1,097.6	414,353	353,745	9.0	302,288	210,202	19.0	8,885
Utah	2,067,154	1,809,358	1,641,901	180.1	414,447	382,741	24.7	16,921	16,921	3.9	25,592
Vermont	924,306	1,014,375	878,708	21.9	53,872	40,740	1.3				4,858
Virginia	3,652,667	3,230,800	3,160,275	989.2	250,441	247,071	35.2	84,278	84,278	4.2	161,043
Washington	3,026,161	3,162,187	2,810,052	163.2	234,814	185,272	1.1				30,836
West Virginia	2,231,412	957,979	954,452	43.6	1,368,548	1,232,649	51.5	47,560	40,180	5.4	4,131
Wisconsin	4,823,884	5,213,259	4,686,736	337.7	133,855	124,000	6.1	5,025	4,900		8,248
Wyoming	2,219,155	2,067,737	2,062,417	139.9	153,801	153,801	12.5				2,937
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	628,269	609,901	8.9	334,873	303,597	8.5				12,535
TOTALS	195,000,000	153,749,822	146,409,958	11,638.7	40,181,083	37,723,948	1,357.7	7,572,365	6,266,635	294.0	4,599,459

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

DEPARTMENT BULLETINS

No. 583D..Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 1279D..Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D..Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T. . .Highway Bridge Surveys. 20 cents.

No. 265T...Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP..The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP.Roadside Improvement. 10 cents.

No. 272MP.Construction of Private Driveways. 10 cents.

No. 279MP.Bibliography on Highway Lighting. 5 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y..Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JULY 31,1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$4,034,617	$2,905,597	$2,905,333	40	1	12	$ 933,721	$ 933,721	8			$ 132,100	$ 132,100	2			$ 63,462
Arizona	1,256,099	1,113,686	1,085,874	13			190,885	153,783	2		3						16,442
Arkansas	3,574,060	2,028,023	2,022,344	40	5	2	1,309,967	1,308,117	15	1		207,825	207,555	1		30	36,045
California	7,486,362	6,297,264	6,062,841	35	8		1,416,095	1,413,189	12			10,000	10,000	5			332
Colorado	2,631,567	1,482,761	1,429,513	19	1		825,499	825,498	8			432,920	364,066	4		21	12,490
Connecticut	1,712,684	141,009	141,009	1	1	1	845,081	823,720	5			719,920	700,735	4			47,220
Delaware	418,239	130,000	130,000	1			277,993	277,993	2								10,246
Florida	2,827,883	1,884,508	1,881,789	21	5		643,879	642,930	9	1							303,163
Georgia	4,895,949	119,097	117,605	3	3	1	1,149,341	1,149,341	22	5		966,910	966,910	21	3	51	2,662,093
Idaho	1,674,479	1,231,709	1,225,244	19	1	8	362,328	362,160	3	1	8	63,913	63,913	1		5	23,162
Illinois	10,307,184	6,277,687	6,256,962	53	6		3,858,805	3,858,805	20	2							191,417
Indiana	5,111,096	2,391,622	2,243,232	25	12		2,846,170	2,846,170	17		163						21,694
Iowa	5,600,679	3,449,125	3,349,076	77	8	7	2,176,654	2,173,501	30	1		86,572	74,590			1	3,512
Kansas	5,246,258	3,110,156	3,105,382	48		5	2,176,281	2,104,036	10	1		111,090	15,000	2			21,841
Kentucky	3,672,387	1,084,167	1,074,334	14	3		2,056,174	1,766,442	10	5		812,339	798,747	4	4		32,864
Louisiana	3,213,467	935,657	935,657	10	1		1,188,906	1,188,897	12	1		1,094,512	892,484	9	1		196,429
Maine	1,426,861	955,109	953,088	18		2	425,417	425,417	2	2		71,740	24,068	1		2	24,288
Maryland	2,061,751	409,070	409,070	3	3	15	807,729	807,729	4	1	4	597,632	525,163	3	1	3	319,790
Massachusetts	4,210,833	1,414,106	1,414,021	12	3		2,323,511	2,323,511	14	1		257,241	257,241	1			216,060
Michigan	6,765,197	4,834,492	4,723,487	40	5		2,065,327	1,994,858	4	3		43,500	43,500		1		3,352
Minnesota	5,395,441	3,936,837	3,863,055	73	11	39	1,501,132	1,475,134	12	2		57,010	57,010			11	242
Mississippi	3,241,475	1,417,521	1,417,356	37	4	1	1,294,268	1,294,268	19	2	1	38,200	38,200			13	491,650
Missouri	6,142,153	1,128,770	1,113,281	17			5,193,850	5,012,959	32	1	1	1,650	1,650			1	14,263
Montana	2,722,327	2,657,848	2,536,243	37	7		239,610	185,931	1	1							153
Nebraska	3,556,441	2,115,893	2,086,102	69	2	12	1,263,645	1,263,645	13	1	6	221,535	196,497	5		6	10,197
Nevada	887,260	690,077	684,294	7	3		211,658	193,432	1			3,630	3,630			5	5,904
New Hampshire	822,484	476,747	476,747	5	4		313,519	313,452	4		1	56,412	32,285	1		6	
New Jersey	3,983,826	959,326	959,326	8	2	1	2,355,029	2,343,984	13	5		743,310	680,516	3	3		
New Mexico	1,725,286	1,572,408	1,568,490	16	1		140,227	140,227	2			56,505	11,202	1			5,367
New York	13,577,189	6,031,240	5,971,216	22	28		7,548,790	7,334,290	23	21		94,000	94,000				177,683
North Carolina	4,823,958	2,651,803	2,651,228	34	17		1,325,407	1,305,856	14	1	108	617,150	617,150	12			249,724
North Dakota	3,207,473	1,898,522	1,894,748	35	2		1,312,725	1,312,725	16	3							
Ohio	8,439,897	1,385,994	1,308,493	7	3	2	5,979,507	5,605,480	38	5	1	1,214,000	1,129,000	12			396,925
Oklahoma	5,004,711	3,061,856	3,054,355	50	6	3	958,256	958,256	9	3	2	1,107,190	974,090	6	3	36	18,010
Oregon	2,334,204	1,786,420	1,780,448	13	6	2	629,664	548,063	4								5,693
Pennsylvania	11,483,613	3,433,545	3,078,062	43	13	9	8,209,274	7,685,841	37	10		473,686	463,301	10			256,409
Rhode Island	699,691	649,511	648,445	4	2		44,314	44,314		1							6,932
South Carolina	3,059,956	1,258,233	1,246,814	25	8	3	1,067,952	1,060,521	18	5	22	372,582	350,782	1	3	23	401,839
South Dakota	3,249,086	1,484,346	1,483,749	33	4	27	1,537,769	1,537,769	30	2	26	281,170	222,213	13			5,356
Tennessee	3,903,979	809,323	801,288	16	2	20	2,588,550	2,588,550	29	1	5	133,610	133,610	1		12	380,531
Texas	10,855,982	8,149,964	8,138,660	110	13	53	2,046,450	2,046,447	20	2	7	258,502	258,502			89	412,373
Utah	1,230,763	568,184	566,779	7	1	1	649,540	649,540	10								14,444
Vermont	729,857	564,414	538,492	7	6	17	1,204,133	187,274	3	1	3						4,090
Virginia	3,774,287	2,286,717	2,176,847	39	14	12	1,035,188	1,028,563	8	4	1	545,970	543,956	3	3	11	24,920
Wasbington	3,095,041	2,469,207	2,434,480	21	11	8	645,949	645,524	2	1	1	4,562	4,562			2	10,476
West Virginia	2,677,937	79,454	79,454	1		4	1,859,691	1,857,723	18	3		664,262	664,262	7	1	3	76,498
Wisconsin	5,022,683	3,600,852	3,566,155	33	5		1,426,581	1,426,537	4	2		68,667	29,367			19	623
Wyoming	1,360,841	665,281	665,173	9			695,669	695,668	5								
Dist. of Columbia	410,804	425,564	410,804	3													
Hawaii	453,703	170,404	170,389	2			351,976	283,314	3								
TOTALS	196,000,000	100,581,106	98,836,834	1275	241	267	81,510,086	78,405,105	597	102	363	12,621,817	11,581,857	133	23	350	7,176,204

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 7 SEPTEMBER 1937

CARS PASSING ON A NARROW ROAD

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 7 September 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

A STUDY OF THE PASSING OF VEHICLES ON HIGHWAYS

By J. T. Thompson, Highway Research Specialist,[1] and Norman Hebden, United States Bureau of Public Roads

THE question of what constitutes suitable or necessary road widths is one of first importance to highway engineers, economists, and administrators. Surface width greatly affects such matters as traffic capacity, highway cost, and safety. In a remarkably short time we have seen widths increase from a scant dozen feet to 20 feet or more for the undivided two-lane pavement and beyond that to multiple-lane arrangements. It is obvious that this increase is the result of the changing character of traffic, but the particular element or elements causing the change—size, speed, or traffic density—has not been determined.

The store of information bearing upon this question is scant—out of proportion to its importance. Various attempts have been made to establish facts, but the investigators have not supplied much of the information needed today in considering the relation of vehicular dimensions and speeds to road widths.

In earlier studies fixed stations were set up on the road at which observers noted the distance from the road edge of vehicles passing the station.[2] Deductions as to the probable transverse positions of vehicles in the most critical state, that is, when passing one another, were accordingly based upon observations involving only one vehicle. It was only by coincidence that simultaneous records of two passing vehicles could be obtained. One exception should be made to this general remark; in the Cleveland study, some data were obtained for passenger cars passing the station simultaneously while traveling in opposite directions.

MOTION PICTURES TAKEN OF PASSING VEHICLES

In the early summer of 1933 the Bureau of Public Roads of the United States Department of Agriculture in cooperation with the Johns Hopkins University, the Commissioner of Motor Vehicles of Maryland, and the State Roads Commission of Maryland,[3] undertook to study this question using a radically different method. It was decided to trail and take motion pictures of vehicles in the act of passing.

The apparatus used in the investigation was simple and needs but little explanation. A motion-picture camera was mounted upon a bracket just outside the driver's window of an automobile as shown in figure 1. This camera was a spring-operated, 35-millimeter machine carrying 100 feet of film at a loading. Exposures were made with a lens having a focal length of 4 inches at the constant rate of 1 foot, or 16 frames, per second. A ratchet-and-pawl arrangement permitted the operator quickly to rewind the camera spring while driving. No serious difficulty was experienced in taking clear pictures.

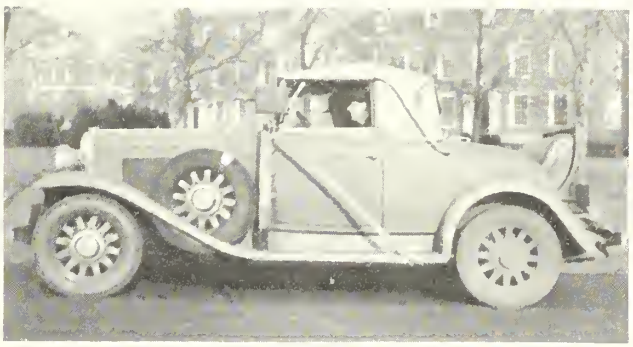

FIGURE 1.—MOTION-PICTURE CAMERA MOUNTED ON AUTOMOBILE USED IN STUDY.

After development, a positive print of the negative was studied in the office by running it through a desk-type, variable-speed machine equipped with a magnifying lens through which the film could be observed as it passed over a translucent plate behind which was a strong light. The frames showing the two vehicles opposite one another in the act of passing were thus identified and marked. (See figs. 2, 3, and 4.) Later, these marked frames were projected upon a screen as still pictures and transverse placement dimensions were scaled off.

It will be helpful to define certain terms that are frequently used in this report.

Critical vehicle—The vehicle being trailed by the observer's car and being passed by another vehicle.

Passing vehicle—The vehicle that passes the critical vehicle.

Lateral position—The transverse position on the road of the vehicles in question when directly opposite one another in the act of passing.

Critical frame—The frame on the film that shows the vehicles at the instant they are opposite each other in the act of passing. This frame is projected to get the required measurements.

Dimension A—The distance from the right edge of the road to the centerline of the right rear wheel of the critical vehicle.

Dimension B—The clearance between the passing and critical vehicles at the instant when their rear wheels are opposite during the act of passing.

Dimension C—The distance from the left edge of the road to the centerline of the outer wheel of the passing vehicle.

Dimension D—The distance center to center of the outer wheels of the passing and critical vehicles.

[1] Also Professor of Civil Engineering, The Johns Hopkins University.

[2] Transverse Distribution of Motor Vehicle Traffic on Paved Highways, by J. T. Pauls, Public Roads, vol. 6, no. 1, March 1925.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio, by the Bureau of Public Roads, 1928.

[3] Besides those already mentioned, other cooperating agencies during 1934 were the Pennsylvania Department of Highways, the Department of Revenue of Pennsylvania, the Commissioner of Motor Vehicles of New Jersey, and the Board of Chosen Freeholders of Union County, N. J.

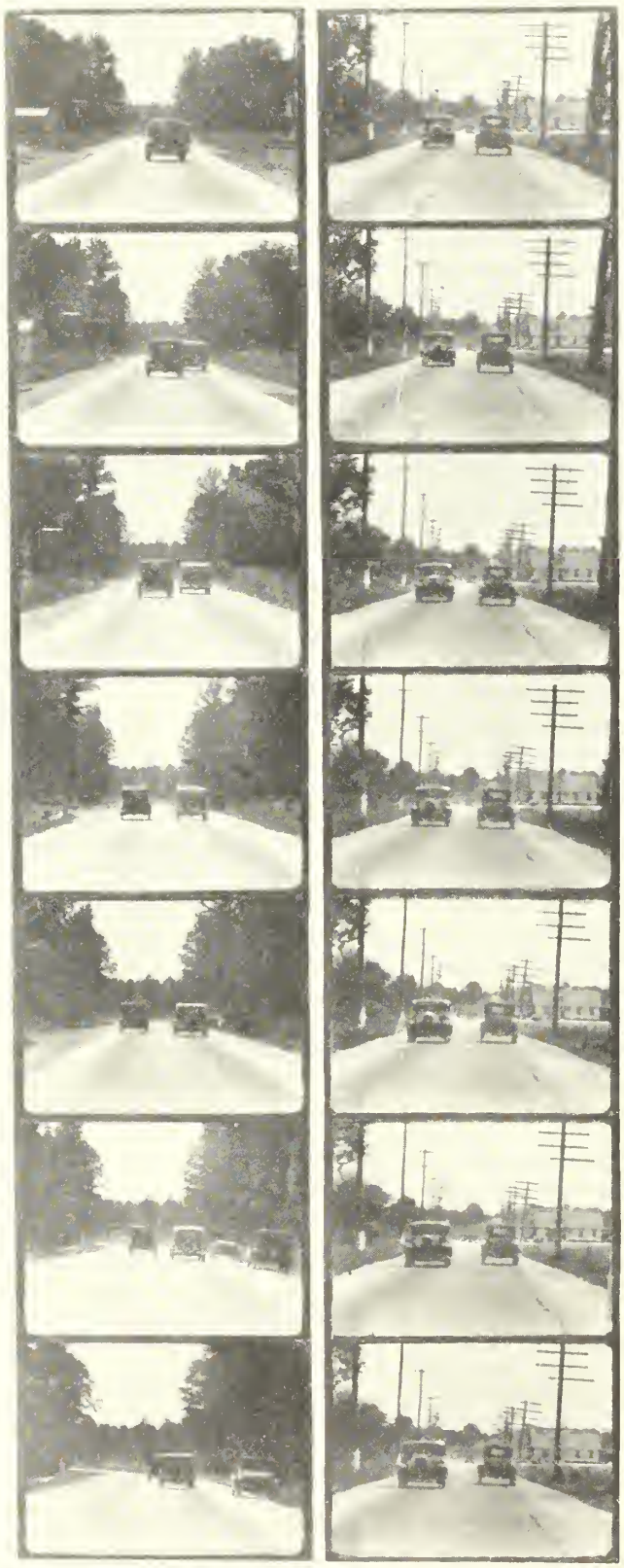

FIGURE 2.—STAGES IN TYPICAL PASSING OPERATIONS. LEFT, PASSENGER CAR PASSING PASSENGER CAR IN THE SAME DIRECTION ON A 20-FOOT ROAD; RIGHT, PASSENGER CAR PASSING PASSENGER CAR IN OPPOSITE DIRECTION ON AN 18-FOOT ROAD.

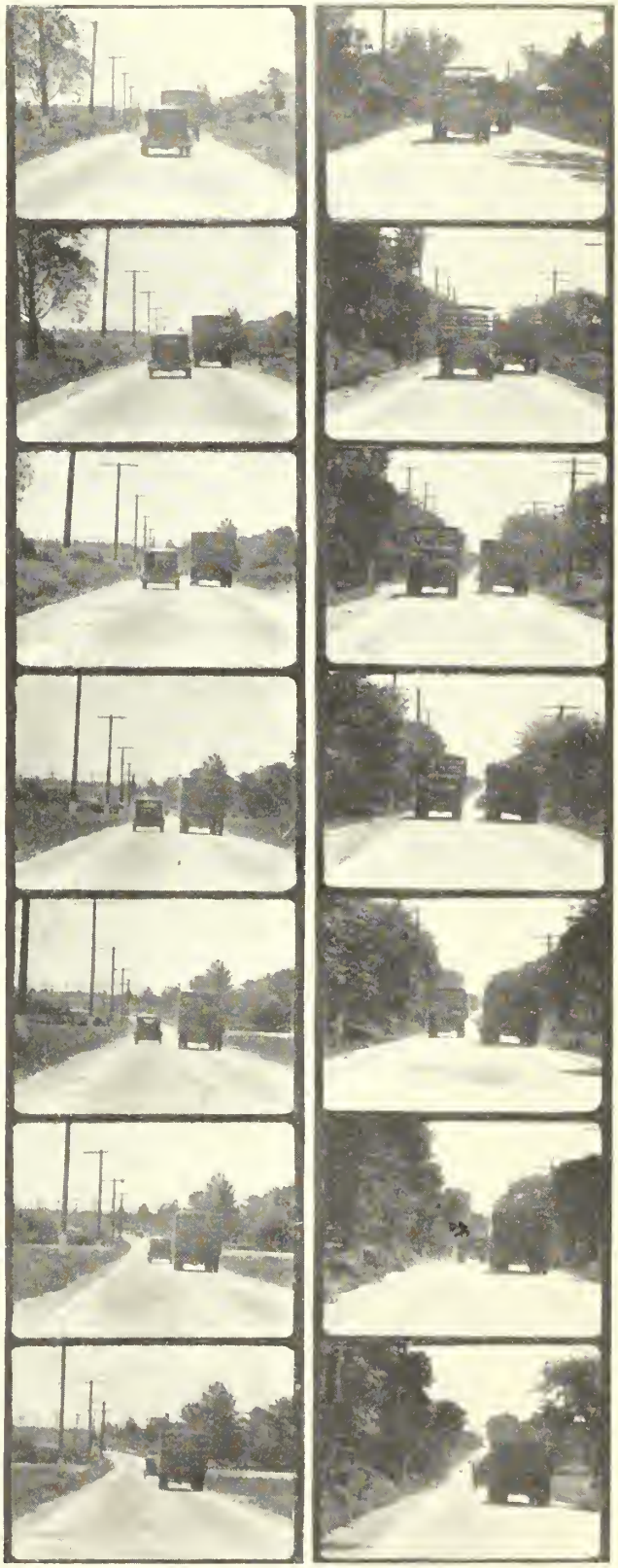

FIGURE 3.—STAGES IN TYPICAL PASSING OPERATIONS. LEFT, PASSENGER CAR PASSING TRUCK IN THE SAME DIRECTION ON AN 18-FOOT ROAD; RIGHT, TRUCK PASSING TRUCK IN THE SAME DIRECTION ON AN 18-FOOT ROAD.

FIGURE 4.—CRITICAL FRAMES OF PASSING OPERATIONS ON 20-FOOT ROADS.

Dimension A+D—The "used space" of road, that is, the distance from the right edge of the road to the centerline of the outer wheel of the passing vehicle.

Dimension E_c—The distance from the right edge of the road to the centerline of the critical vehicle.

Dimension E_p—The distance from the left edge of the road to the centerline of the passing vehicle.

Offset—Distance between centerline of traffic lane and center of vehicle, negative when measured from the lane center toward the road edge, positive when otherwise.

The dimensions defined above apply to vehicles passing while traveling in the same or opposite directions and with the exception of offsets, E_c, and E_p are shown diagrammatically in figure 5.

DISTANCES SCALED FROM ENLARGED PICTURES

Table 1 shows a sample of the data recorded in the field and information derived in the office. The field procedure was as follows: The observers placed their car in free traffic and selected a vehicle for observation and followed 200 to 300 feet behind it—near enough to get a useful picture but sufficiently far away to encourage a third vehicle to pull in between. Just as the middle vehicle pulled out to go around the leading one, the camera was started by the driver-observer and a picture of the entire passing maneuver was taken. The observers' car was kept as nearly as possible at the speed of the critical vehicle.

Tests were made in advance of the field work to determine how accurately the trailing speed would represent the speed of the vehicle trailed and it was found that the greatest error over a wide range of speeds did not exceed 5 miles per hour. The speedometer reading was recorded by a second observer who also noted, from stakes set at one-tenth mile intervals, the approximate point of passing. This observation led to a close identification of the point and subsequently notes were made regarding the dimensions of the road, the characteristics of its surface, the shoulders, and wayside conditions.

When the critical vehicle was a truck, it was stopped after the pictures were taken, and its over-all length, width, and distance center to center of tire mountings were measured. No attempt was made to stop the passing vehicle. When the critical vehicle was a passenger car, it was not stopped for measurement because it was felt that for all practical purposes such dimensions could be considered constant.

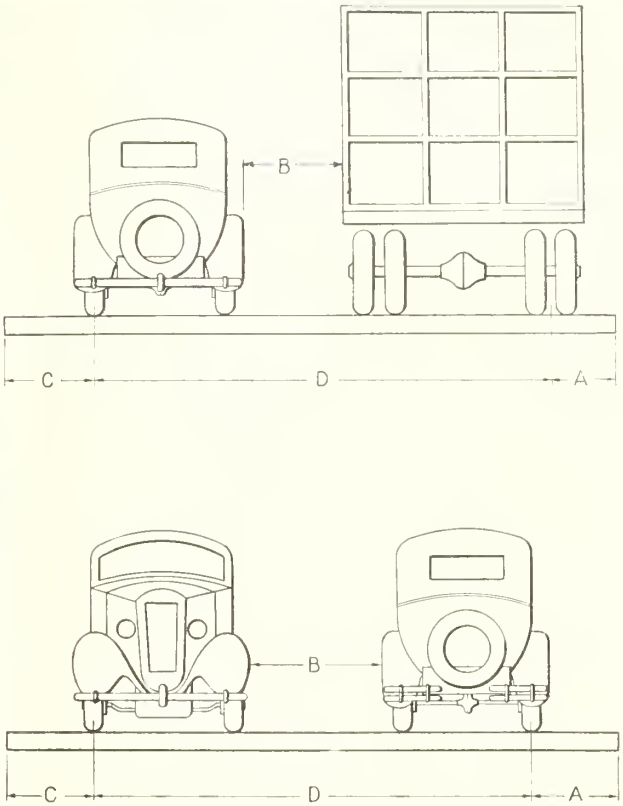

FIGURE 5.—DIMENSIONS USED IN DISCUSSING VEHICLES, PASSING IN SAME AND OPPOSITE DIRECTIONS.

In obtaining data on vehicles passing from opposite directions, only sufficient film was exposed to determine the lateral positions of the vehicles at the instant of passing. A record of the speed of the critical vehicle and the point of passage was also made.

Positive prints of film were run through a machine designed for use in film editing for the purpose of correlating film "shots" and field notes and selecting the critical frames. At this time decision was made regarding the usefulness of the picture and all observations where the positions were affected by special conditions, such as vehicles parked upon the shoulder or people walking along the side of the road, were eliminated from further consideration.

After the acceptable critical frames had been selected, they were projected upon a screen and the desired dimensions scaled off. Wherever possible, use was made of the known width of the road to establish the scale of the other dimensions. These scaled values were recorded and converted into actual position dimensions as shown in the samples in table 1.

Considerable thought was given the matter of accuracy and tolerance in scaling the dimensions. To insure the best possible accuracy and to act as a check, two different observers made measurements of the clearance, B, on every critical frame. This check and a comparison of A+C+D against the road width as measured in the field, was made in every case to insure accuracy. The tolerances adopted allowed a variation of 0.3 foot in the measurements of clearance, B, and also between the sum of A+C+D and the measured road width. These tolerances amounted to approxi-

TABLE 1.—*Samples of field data and data derived in office*

SAMPLE OF FIELD DATA, LEFT PAGE OF NOTEBOOK

Location: Philadelphia Road Date: July 25, 1933 Party {N. H. W. M.

Serial no.	Weather	Aperture of camera	Passing vehicle: Type	Direction	Approximate station, tenths of miles (maintenance stakes)	Speed miles per hour	Critical vehicle: License no.	Type	Length	Width	Center to center of mountings
									Feet	*Feet*	*Feet*
S-145	Bright	15	Passenger	N	35¾ S – 36¾ N	20	Md. 764-T	4 WDT and 2 WDSTr	45.9	8.0	5.95
O-17	do	15	do	N	51¼	40	do	do	45.9	8.0	5.95
S-146	Hazy	13	do	S	17½ N – 15¾ S	35		Passenger			

SAMPLE OF FIELD DATA, RIGHT PAGE OF NOTEBOOK

Serial no.	Point of passing	Road: Type	Over all width	Condition of surface	Paved shoulder: Width	Paved shoulder: Condition	Dirt shoulder: Width	Dirt shoulder: Condition	Remarks on wayside conditions
			Feet		*Feet*		*Feet*		
S-145	36+205	Concrete 15.0	18.0	Rough, patched	R. N. 3 concrete	Fair	R. N. 3	Bad	Deep ditch at 5 feet from road edge.
							L. N. 5	Fair	Deep ditch at 5 feet.
O-17	51+117	do	18.0	Fair	do	do	R. N. 3	Poor	Ditch at 3 feet.
							L. N. 4	do	Ditch at 6 feet, bad edge.
S-146	16+325	Sheet asphalt	20.0	Smooth	2 each 3 feet concrete	Good	R. S. 6	Bad	Ditch at 7 feet.
							L. S. 4	Fair	Mail boxes at 4 to 5 feet.

SAMPLE OF OFFICE DATA

Date: July 3, 1933 Computer: J. J.

Serial no.	Reference	Reference: Actual width	Reference: Projected width	Width of critical vehicle: Projection	Width of critical vehicle: Actual	Width of passing vehicle: Projection	Width of passing vehicle: Actual	Projected distances: A	Projected distances: B	Projected distances: C	Projected distances: D	Actual distances: A	Actual distances: B	Actual distances: C	Actual distances: D
		Feet	*Inches*	*Inches*	*Feet*	*Inches*	*Feet*	*Inches*	*Inches*	*Inches*	*Inches*	*Feet*	*Feet*	*Feet*	*Feet*
S-294	Over-all road width	20.0	9.9	2.9	5.8	2.9	5.8	1.4	2.0	1.3	7.2	2.8	4.0	2.6	14.5
O-337	do	22.0	10.7	2.9	6.0	2.8	5.9	.7	3.6	1.4	8.6	1.3	7.3	2.9	17.8
S-308	do	18.0	13.9	4.6	5.9	4.5	5.8	1.2	3.2	1.3	11.4	1.6	4.1	1.7	14.7

mately 5 percent for the clearance and 2 percent for the sum of A+C+D. A larger tolerance was allowed in the measurement of the clearance because this dimension was the most difficult to scale because of the indistinct outline of the vehicles in the projection when inspected at close range.

PASSING A VEHICLE GOING IN SAME DIRECTION MAKES GREATEST DEMAND FOR ROAD WIDTH

The data obtained are sufficient to indicate the habits of drivers in passing other vehicles going in the same direction and in opposite directions on roadways of widths ranging from insufficient to ample. Widths of 18, 20, and 22 feet, were thought to give such a range. All pictures were taken on undivided, primary highways carrying recreational and commercial traffic. In general, the passing of vehicles was recorded on roads without paved shoulders but some studies were made on roads widened by shoulder paving. Table 2 gives the type, width, and shoulder conditions, on each of the roads where studies were made.

A few of the roads on which observations were made had center stripes painted on the surface to mark the lanes. Most of the concrete roads on which observations were made had longitudinal center joints that also served to mark the common boundary of the two lanes at the center of the road. No effect of the presence of a center stripe upon the position taken by either the critical or the passing vehicle during passing operations was observed.

TABLE 2.—*Description of roads on which observations were made*

Width (feet)	Route no. and location	Year of observation	Description	Dirt shoulder
18	U. S. 40, vicinity of Aberdeen, Md.	1933–34	Concrete	Poor, 1 to 3 feet wide.
18	U. S. 111, Maryland line to York, Pa.	1934	do	Do.
18	U. S. 22, vicinity of Allentown, Pa.	1934	do	Do.
18	U. S. 40, Baltimore to Aberdeen, Md.	1933	15-foot concrete road, widened with a 3-foot concrete strip on 1 side.	Poor, 1 to 4 feet wide.
20	U. S. 40, vicinity of Baltimore, Md.	1933	Bituminous concrete with a 3-foot concrete strip on each side.	In built-up section, very narrow.
20	U. S. 40, vicinity of Aberdeen, Md.	1933–34	Concrete	Fair, 5 to 8 feet wide.
20	U. S. 22, vicinity of Allentown, Pa.	1934	do	Poor, 3 to 6 feet wide.
22	U. S. 40, vicinity of Baltimore, Md.	1933	16-foot asphalt with a 3-foot concrete strip on each side.	Poor, 2 to 4 feet wide.
22	U. S. 111, vicinity of Baltimore, Md.	1933	16-foot bituminous concrete with a 3-foot concrete strip on each side.	Fair, 4 to 8 feet wide.
22	Westfield Ave., vicinity of Rahway, N. J.	1934	Concrete	Excellent, 12 feet wide.

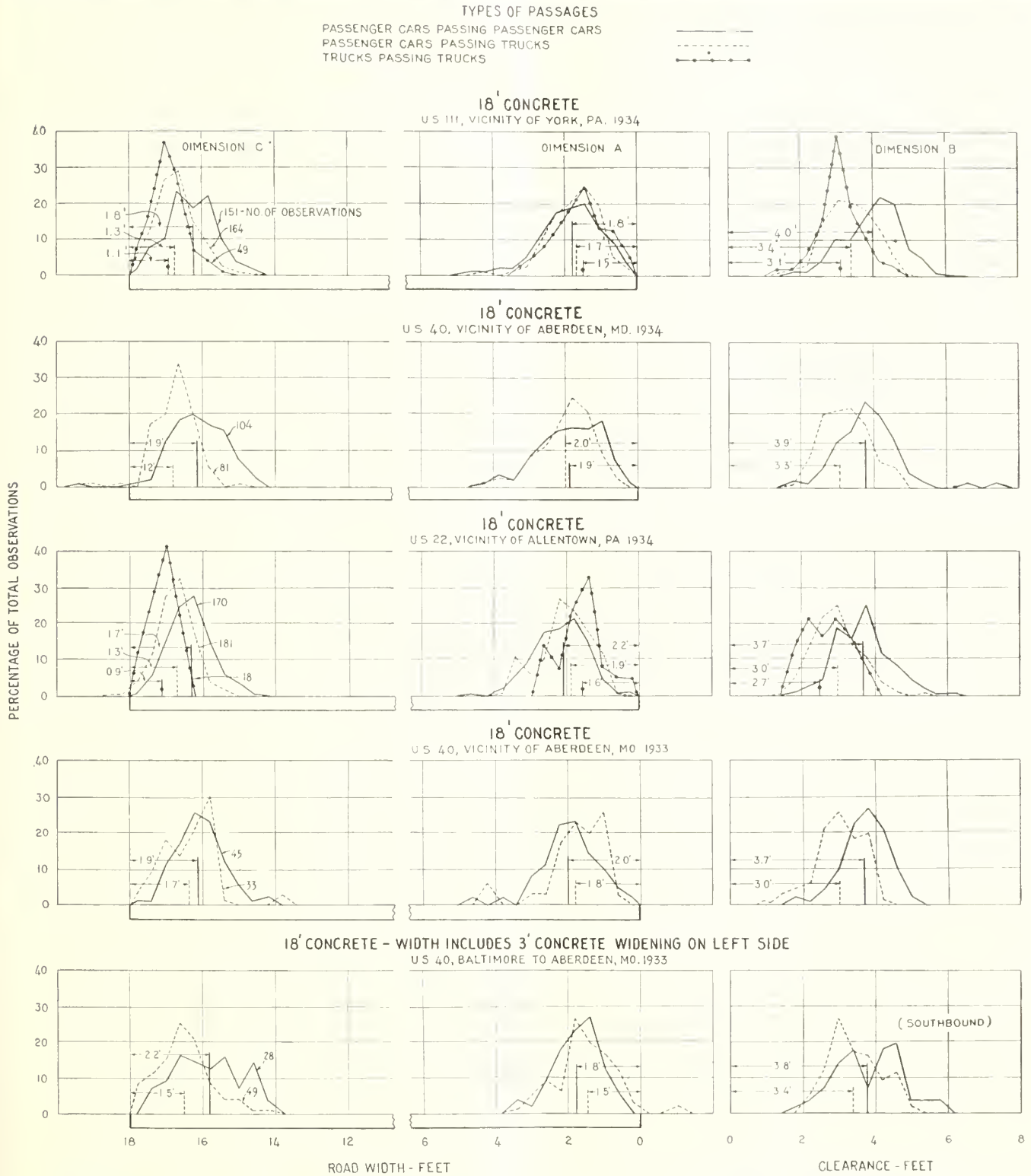

FIGURE 6.—FREQUENCY DISTRIBUTION OF DIMENSIONS A, B, AND C (SEE FIG. 5) FOR SAME-DIRECTION PASSING. NUMBER OF OBSERVATIONS IN EACH SAMPLE INDICATED BY NUMBER AGAINST FREQUENCY DISTRIBUTION LINE.

Overtaking and passing a vehicle going in the same direction is a more difficult operation and imposes a greater demand for road width than meeting and passing a vehicle. In meeting an oncoming vehicle a driver selects a position within the right lane and makes sure that the oncoming vehicle does not tend to infringe upon his lane. Experience has taught that this is the best method to avoid sideswiping. Speed can be regulated according to local conditions. As will be pointed out later, passenger cars do not run off the pavement when passing other passenger cars as is sometimes the case in same-direction passing. In same-direction passing the driver must use that portion of the roadway left to him by the vehicle ahead, dividing his attention

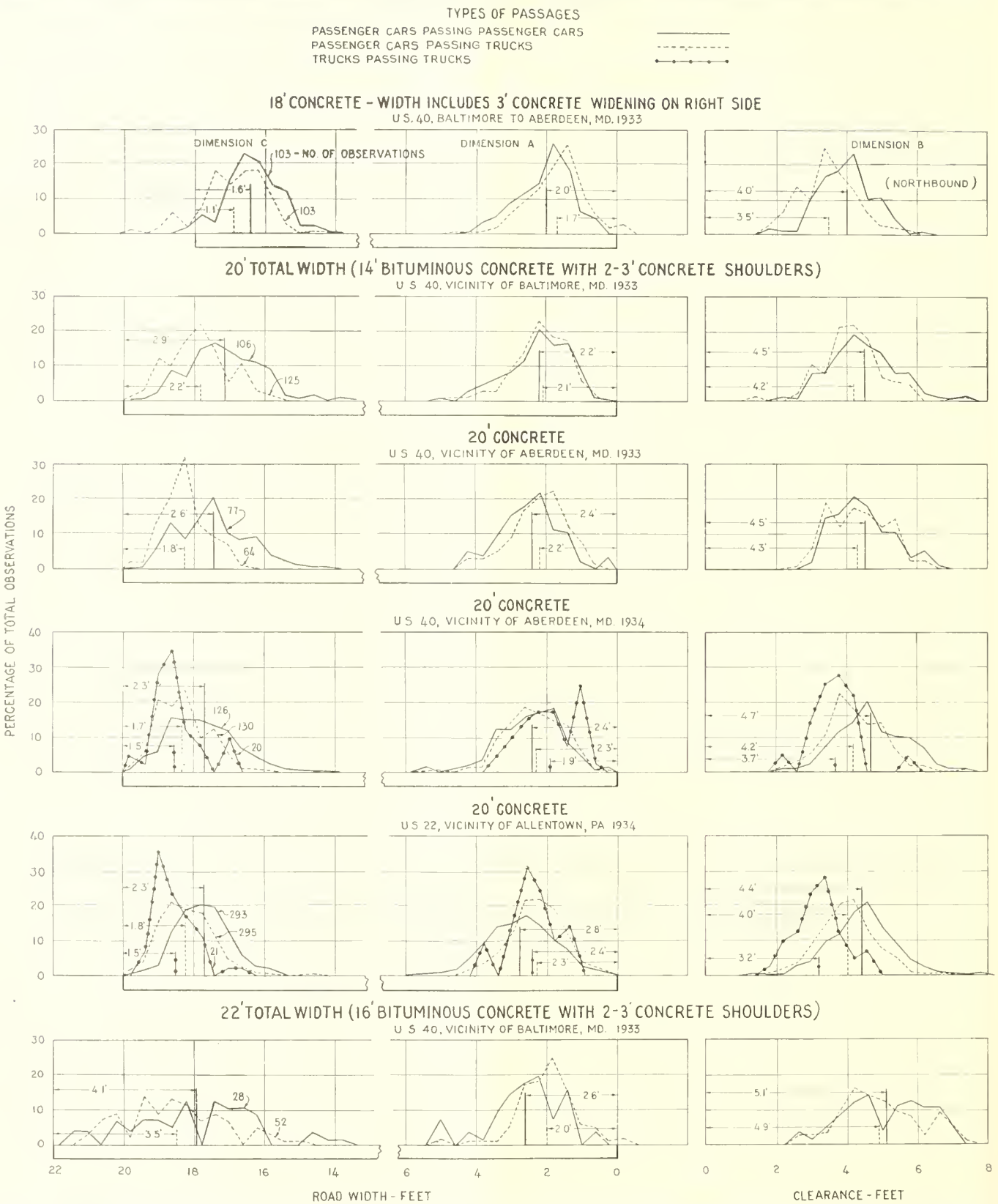
TYPES OF PASSAGES
PASSENGER CARS PASSING PASSENGER CARS
PASSENGER CARS PASSING TRUCKS
TRUCKS PASSING TRUCKS
18' CONCRETE - WIDTH INCLUDES 3' CONCRETE WIDENING ON RIGHT SIDE
U.S. 40, BALTIMORE TO ABERDEEN, MD. 1933
DIMENSION C
103 - NO. OF OBSERVATIONS
DIMENSION A
DIMENSION B
(NORTHBOUND)
20' TOTAL WIDTH (14' BITUMINOUS CONCRETE WITH 2-3' CONCRETE SHOULDERS)
U S 40, VICINITY OF BALTIMORE, MD. 1933
20' CONCRETE
U S 40, VICINITY OF ABERDEEN, MD. 1933
20' CONCRETE
U S 40, VICINITY OF ABERDEEN, MD. 1934
20' CONCRETE
U S 22, VICINITY OF ALLENTOWN, PA 1934
22' TOTAL WIDTH (16' BITUMINOUS CONCRETE WITH 2-3' CONCRETE SHOULDERS)
U S 40, VICINITY OF BALTIMORE, MD. 1933
PERCENTAGE OF TOTAL OBSERVATIONS
ROAD WIDTH - FEET
CLEARANCE - FEET

FIGURE 7.—FREQUENCY DISTRIBUTION OF DIMENSIONS A, B, AND C (SEE FIG. 5) FOR SAME-DIRECTION PASSING. NUMBER OF OBSERVATIONS IN EACH SAMPLE INDICATED BY NUMBER AGAINST FREQUENCY DISTRIBUTION LINE.

between clearance with the vehicle on the right and the road edge on the left, and must travel at a speed greater than that of the vehicle being passed.

Figures 6, 7, and 8 show, for same-direction passing, the frequency distribution of edge distance of the vehicles being passed (dimension A), the edge distance

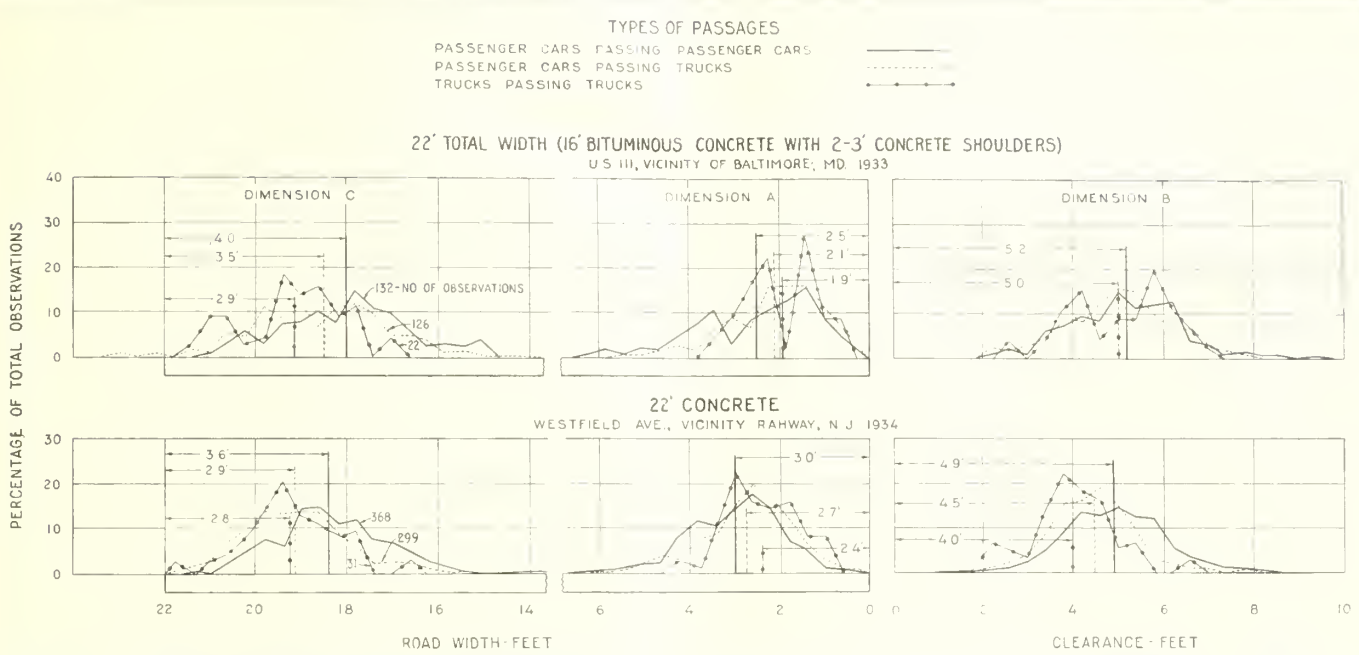

FIGURE 8.—FREQUENCY DISTRIBUTION OF DIMENSIONS A, B, AND C (SEE FIG. 5) FOR SAME-DIRECTION PASSING. NUMBER OF OBSERVATIONS IN EACH SAMPLE INDICATED BY NUMBER AGAINST FREQUENCY DISTRIBUTION LINE.

of the passing vehicles (dimension C), and the clearance between vehicles (dimension B), for each of the roads where studies were made. Passenger cars passing passenger cars are reported separately from passenger cars passing trucks.

In some instances data for trucks passing trucks are shown. Special effort was made to record trucks passing trucks but the number of observations was small, amounting to less than 6 percent of the overtaking passages recorded. This small percentage probably is the result of the relatively small proportion of trucks to total traffic and possibly to the absence of pronounced differences in speed among trucks.

Examination of figures 6 to 8 shows only slight differences in the average positions of vehicles on roads of the same width. For the 18-foot surfaces the frequency distribution lines for dimensions A, B, and C are approximately triangles with narrow bases and high altitudes. With increase in surface width to 20 feet the peaks are somewhat flattened and the bases spread out and this effect is very much more pronounced for 22-foot surfaces. This change in shape of the diagrams is an indication of relief from road-cramping.

Average dimensions from the diagrams for same-direction passing and also those for opposite-direction passing to be presented later, are given in table 3. There is surprisingly little variation in the average dimensions for surfaces of the same width, seldom more than one-half foot. This is about the width of a passenger-car tire and gives confidence as to the adequacy of the methods used.

Table 4 shows the average dimensions consolidated for each width of road but excludes bituminous roads with concrete shoulders and one concrete road widened with a 3-foot strip of concrete. This was done to eliminate the possible influence of paved shoulders on vehicle position. The table is based entirely on observations on 18-, 20-, and 22-foot concrete pavements without special shoulder construction and all conclusions as to vehicle positioning are based upon these consolidated data.

DRIVER PSYCHOLOGY AND RELATION OF ROAD WIDTH TO VEHICLE POSITION INDICATED BY DATA

Table 4 throws light upon several moot questions. For example it has been thought that, perhaps because truck drivers have greater experience and are aware that their vehicles are generally of such width as to cause inconvenience to others, they keep closer to the right edge of the road than do passenger-car operators. Obviously this is not the case as both passenger cars and trucks apparently tend to center themselves closely on the centerline of their own traffic lane and maintain that position when being overtaken and passed. This seems to be true indiscriminately for all three of the road widths studied as the dimensions E_c and the corresponding offsets of critical vehicles show.

One also wonders what drivers want or try to do, either consciously or subconsciously, when they overtake and pass other vehicles. Do they follow the centerline of their own traffic lane if they can? Are they equally concerned with the danger of sideswiping the vehicle they are passing and the hazard of running off onto the left shoulder, and as a result do they bisect the clear space between the vehicle and the road edge?

The answer to the first question is not entirely clear from the data of table 4. In contrast to the positioning of the critical vehicle whose average offset is never greater than 0.2 foot and which is alternately plus and minus, the passing vehicle is consistently to the left of its lane center, except when the relatively small passenger cars are alone involved on the relatively wide 22-foot pavement. In this case the passing vehicles could obviously follow the lane center if they wanted to but instead they apparently are satisfied with a clearance of about 5.0 feet and move well inside the lane centerline.

The answer to the second question seems to be that they are more afraid of sideswiping, since in every case they pass well to the left of the midpoint between critical vehicle and left road edge. This is brought out strikingly by figure 9 which shows diagramatically the positions of the critical and passing vehicles with respect

TABLE 3.—*Average dimensions on individual roads for various types of passages; vehicles moving in either the same or opposite direction*

Road width, feet	Description	Year of observation	Same-direction passing: Passenger cars passing passenger cars: Number of observations	C	E_p	B	A	E_o	Used space	Same-direction passing: Passenger cars passing trucks: Number of observations	C	E_p	B	A	E_o	Used space	Same-direction passing: Trucks passing trucks: Number of observations	C	E_p	B	A	E_o	Used space	Opposite-direction passing: Passenger cars passing passenger cars: Number of observations	A or C	E_o or E_p	Number of observations	B	Opposite-direction passing: Trucks passing trucks: Number of observations	A or C	E_o or E_p	Number of observations	B
	18-FOOT SURFACES																																
				Ft.	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*		*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*			*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*			*Ft.*	*Ft.*		*Ft.*			*Ft.*		
18	U S 40, Baltimore-Aberdeen, Md., north-bound, concrete with 3-foot concrete strip on east side	1933	103	1.6	4.0	4.0	2.0	4.4	16.4	103	1.1	3.2	3.5	1.7	4.5	17.0								74	1.9	4.2	37	3.9	38	0.9	3.7	19	3.0
18	U S 40, Baltimore-Aberdeen, Md., south-bound, concrete with 3-foot concrete strip on east side	1933	28	2.2	4.5	3.8	1.8	4.1	15.9	49	1.5	3.6	3.4	1.5	4.3	16.6																	
18	U S 40, vicinity Aberdeen, Md., concrete	1933	45	1.9	4.3	3.7	2.0	4.4	16.2	33	1.7	4.0	3.0	1.8	4.5	16.4								20	1.8	4.2	10	3.8					
18	do	1934	104	1.9	4.2	3.9	1.9	4.2	16.1	81	1.2	3.3	3.3	2.0	4.7	16.8								294	1.8	4.1	147	4.1	128	1.1	4.0	64	2.8
18	U S 111, Maryland line to York, Pa., concrete	1934	151	1.8	4.1	4.0	1.8	4.2	16.3	164	1.3	3.5	3.4	1.7	4.4	16.7	49	1.1	3.5	3.1	1.5	4.2	16.9	398	1.8	4.1	199	4.1	404	1.1	3.9	202	3.1
18	U S 22, vicinity Allentown, Pa., concrete	1934	170	1.7	4.0	3.7	2.2	4.5	16.3	181	1.3	3.5	3.0	1.9	4.8	16.7	18	.9	3.3	2.7	1.6	4.5	17.1	110	2.0	4.4	55	3.5	138	1.2	4.0	69	2.6
	20-FOOT SURFACES																																
20	U S 40, vicinity Baltimore, Md., bituminous concrete with 3-foot concrete strip on each side	1933	106	2.9	5.2	4.5	2.2	4.6	17.1	125	2.2	4.2	4.2	2.1	4.9	17.9								54	2.5	4.8	27	4.8	58	1.6	4.3	29	4.4
20	U S 40, vicinity Aberdeen, Md., concrete	1933	77	2.6	5.0	4.5	2.4	4.8	17.4	64	1.8	4.0	4.3	2.2	4.9	18.2																	
20	do	1934	126	2.3	4.6	4.7	2.4	4.8	17.7	130	1.7	3.9	4.2	2.3	5.1	18.3	20	1.5	4.0	3.7	1.9	4.7	18.5	452	2.2	4.6	226	5.1	172	1.6	4.5	86	3.7
20	U S 22, vicinity Allentown, Pa., concrete	1934	293	2.3	4.7	4.4	2.8	5.2	17.7	295	1.8	4.1	4.0	2.3	5.1	18.2	21	1.5	3.9	3.2	2.4	5.3	18.6	404	2.5	4.9	202	4.5	324	1.7	4.6	162	3.4
	22-FOOT SURFACES																																
22	U S 40, vicinity Baltimore, Md., asphalt with 3-foot concrete strip on each side	1933	28	4.1	6.4	5.1	2.6	4.9	18.0	52	3.5	5.5	4.9	2.0	4.8	18.5																	
22	U S 111, vicinity Baltimore, Md., bituminous concrete with 3-foot concrete strip on each side	1933	132	4.0	6.4	5.2	2.5	4.9	18.2	126	3.5	5.5	5.0	2.1	4.8	18.7	22	2.9	5.0	5.0	1.9	4.7	19.4	132	2.5	4.7	66	6.8	78	1.8	4.4	39	6.1
22	Westfield Ave. —vicinity Rahway, N. J., concrete	1934	368	3.6	6.0	4.9	3.0	5.3	18.4	299	2.9	5.2	4.5	2.7	5.6	19.1	31	2.8	5.3	4.0	2.4	5.4	19.2	496	2.9	5.3	248	5.7	344	2.2	5.2	172	4.3

TABLE 4.—*Weighted average distances on physically similar concrete roads (without paved shoulder) for various types of passages, vehicles moving in either the same or opposite direction*

PASSENGER CARS PASSING PASSENGER CARS

Road width (feet)	Vehicles moving in same direction			Vehicles moving in opposite direction			Vehicles moving in same direction	Vehicles moving in opposite direction	Vehicles moving in same direction			Vehicles moving in opposite direction			Space used by vehicles moving in same direction (A+D)	
	C	E_p	Offset	C	E_p	Offset	B	B	A	E_c	Offset	A	E_c	Offset	A+D	Percentage of road width
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Percent*
18	1.8	4.1	−0.4	1.8	4.2	−0.3	3.8	4.0	2.0	4.3	−0.2	1.8	4.2	−0.3	16.2	90.0
20	2.3	4.7	−0.3	2.4	4.7	−0.3	4.5	4.8	2.6	5.0	0	2.4	4.7	−0.3	17.7	88.5
22	3.6	6.0	+0.5	2.9	5.3	−0.2	4.9	5.7	3.0	5.3	−0.2	2.9	5.3	−0.2	18.4	83.6
PASSENGER CARS PASSING TRUCKS																
18	1.3	3.5	−1.0				3.2		1.8	4.6	+0.1				16.7	92.7
20	1.8	4.0	−1.0				4.1		2.3	5.1	+0.1				18.2	91.0
22	2.9	5.2	−0.3				4.5		2.7	5.6	+0.1				19.1	86.9
TRUCKS PASSING TRUCKS																
18	1.0	3.4	−1.1	1.1	3.9	−0.6	3.0	2.9	1.6	4.3	−0.2	1.1	3.9	−0.6	17.0	94.5
20	1.5	3.9	−1.1	1.7	4.2	−0.8	3.5	3.5	2.2	5.0	0	1.7	4.2	−0.8	18.5	92.5
22	2.8	5.3	−0.2	2.2	5.2	−0.3	4.0	4.3	2.4	5.4	−0.1	2.2	5.2	−0.3	19.2	87.1

to the centerlines of traffic lanes and the position of the passing vehicle with respect to the midpoint referred to. Figure 9 is based upon the consolidated data of table 4.

Reference has previously been made to the shape of the distribution diagrams of figures 6, 7, and 8 as an index to the relief from road cramping that is experienced as road widths increase. The reduction in the height of the peaks and the increase in the width of the bases is not nearly so marked between the 18- and 20-foot as between the 20- and 22-foot surfaces.

Other evidence of the greater convenience of traffic on the wider roads also appears in figures 6 to 8. Passenger cars when passing other passenger cars on 18-foot roads were observed in a number of instances to run with their left wheels on the dirt shoulder. This did not happen on either of the two wider roads.

When the average positions of passing vehicles are studied in table 4 or figure 9 very little if any relief from cramping is apparent when the road width increases from 18 to 20 feet. Passenger cars when passing passenger cars can reduce their offset 0.1 foot but there is no change in the offsets of passing vehicles when passenger cars pass trucks or when trucks pass trucks. However, when the road width increases to 22 feet, there is a marked reduction of offsets and all types of vehicles seem to be much more comfortably accommodated. As far as offsets are concerned trucks, when passing trucks on the 22-foot pavement, are able to assume positions at least as favorable as passenger cars passing passenger cars on the 20-foot road and more favorable positions as far as edge distance with respect to the left wheels is concerned.

Figures 10 and 11 show frequency distributions similar to those of figures 6, 7, and 8, except that vehicles are moving in opposite directions. The change in shape of diagrams with increase in road width has a similar significance. It should be noted that no passenger cars were observed to run off on to the dirt shoulder as was the case when they were overtaking and passing other passenger cars on the 18-foot pavement.

The edge distances, positions of vehicle centers, and offsets are also shown for opposite direction passing in table 4. The offsets on all roads and for both types of vehicles are consistently negative. It may be concluded that this displacement to the driver's right is influenced by the presence of the oncoming vehicle since in same direction passing, critical vehicles on the average were seen to track in the center of their traffic lane.

From the foregoing it may be concluded that a pavement width of 18 feet is too narrow for either passenger cars alone or mixed traffic, that pavements 20 feet wide are inadequate for dense traffic involving wide trucks but are reasonably satisfactory for the more lightly traveled roads and for roads used infrequently by wide trucks, and that a width of 22 feet is entirely adequate and satisfactory for mixed traffic.

Speeds of all types of vehicles have steadily increased in the past and there is no definite assurance as to the future trend. It is believed that speed has an effect upon the position of motor vehicles on the pavement. A limited study was made to show the effect of speed upon the position of passenger cars relative to the right edge of the road. Frequency distribution diagrams for same-direction passing were drawn, as shown in figure 13, and the average position with respect to the right road edge was determined for the various speed groups. These positions were taken by the cars as they were being passed by other passenger cars on Westfield Avenue near Rahway, N. J. It is a 22-foot concrete highway. As the speed of the critical vehicle increases, its distance from the right road edge is increased. Additional curves for higher speeds were plotted and they show the same trend, but they are not presented because of the limited number of observations made.

It is felt that, of any effects speed may have upon vehicle position, the primary one is that involving greater edge distance. Thus, further increase in the speeds of vehicles will tend to make additional road width necessary.

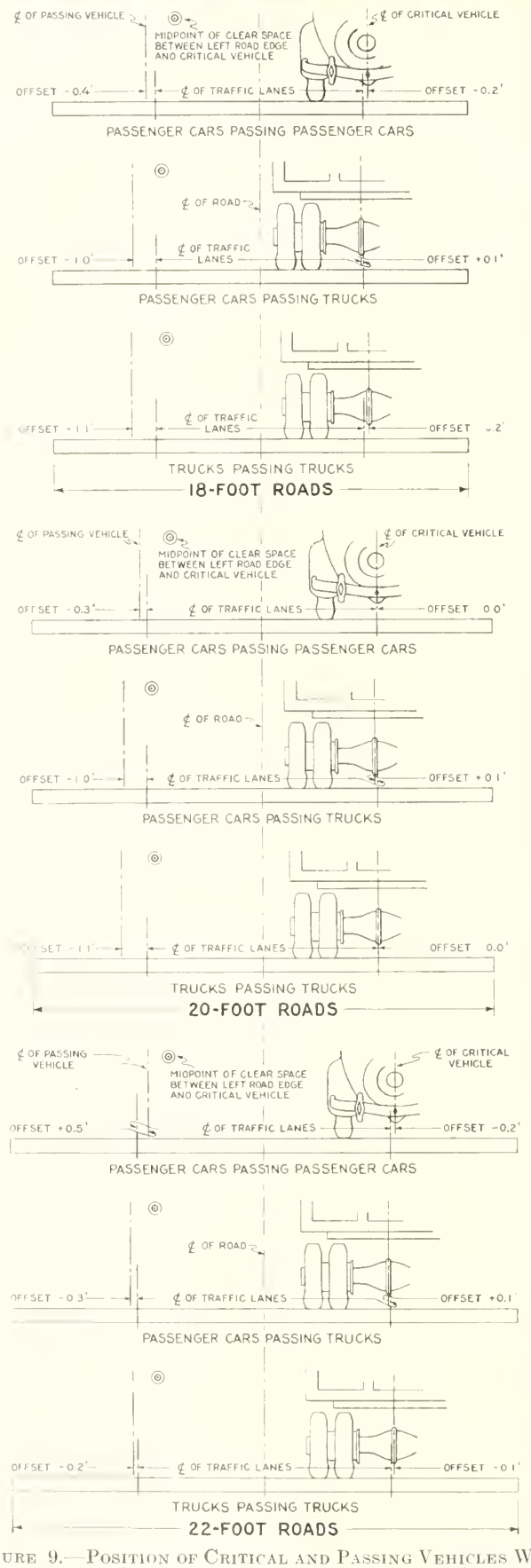

FIGURE 9.—POSITION OF CRITICAL AND PASSING VEHICLES WITH RESPECT TO CENTERS OF TRAFFIC LANES AND CENTER OF MIDPOINT OF CLEAR SPACE BETWEEN LEFT ROAD EDGE AND CRITICAL VEHICLE FOR SAME DIRECTION PASSING. DIMENSIONS ARE WEIGHTED AVERAGES FOR PHYSICALLY SIMILAR CONCRETE ROADS. (WITHOUT PAVED SHOULDERS.)

An interesting and rather surprising fact brought out by this study is the magnitude of the clearances taken by motor vehicles. This holds true for all types of passages whether in same-direction or opposite-direction passing. Quite contrary to the feeling that drivers often have of "just getting by" when they pass other vehicles, the large average clearances observed show that this feeling is generally unwarranted. The suggestion often made that narrow roads would be satisfactory were traffic composed solely of passenger cars is based on the fact that with small edge distances and clearances it is physically possible for vehicles to pass. The facts determined in this study definitely indicate that fairly large edge distances and clearances are desired by vehicle drivers.

TRUCKS CAUSE SMALL INCREASE IN USED WIDTH OF SURFACE

Information on the influence of truck width upon the used width of highways (A+D) has been sought by those studying the allocation of highway costs to the various classes of vehicles. In order to bring out facts in this connection diagrams were drawn for cases in which passenger cars overtook and passed trucks. Each observed A+D dimension was plotted against the corresponding overall width of the critical truck. These data are shown in figure 12. It will be observed that the bulk of the data lies within the 7- to 8-foot range of truck widths, and that outside this range the points become fewer and more scattered. With this observation in mind, and because a recent survey shows approximately two-thirds of all trucks to have widths between 7 and 8 feet,[4] the method of least squares was applied to the data within this range only, to determine the average line.

This analysis is summarized in table 5 in which the increase in used space for a 1-foot increase in truck width is recorded. The results are quite variable but on the average clearly indicate that as truck widths increase, passing passenger cars shift further toward the left edge. The amount they shift, however, is small, 0.1 foot on

TABLE 5.—*Summary showing increase in used space for increase in truck width from 7 to 8 feet for passenger cars passing trucks on concrete roads*

Location of road	Road width	Year of observation	Number of observations	Used space (A+D)		Increase in A+D for 1 foot increase in truck width
				Truck width 7 feet	Truck width 8 feet	
	Feet			*Feet*	*Feet*	*Feet*
U S 40, vicinity Aberdeen, Md.	18	1933	24	16.29	16.57	0.28
U S 40, vicinity Aberdeen, Md.	18	1934	57	16.54	16.82	.28
U S 111, Maryland line to York, Pa.	18	1934	111	16.63	16.76	.13
U S 22, vicinity Allentown, Pa.	18	1934	169	16.73	16.74	.01
Weighted average for 18-foot road				16.64	16.75	.11
U S 40, vicinity Aberdeen, Md.	20	1933	56	17.85	18.51	.66
U S 40, vicinity Aberdeen, Md.	20	1934	98	18.23	18.37	.14
U S 22, vicinity Allentown, Pa.	20	1934	267	17.94	18.29	.35
Weighted average for 20-foot road				18.00	18.34	.34
Westfield Avenue, vicinity Rahway, N. J.	22	1934	147	18.80	19.46	.66

[4] A Study of the Weights and Dimensions of Trucks by J. T. Thompson, Public Roads, vol. 16, no. 3, May 1935.

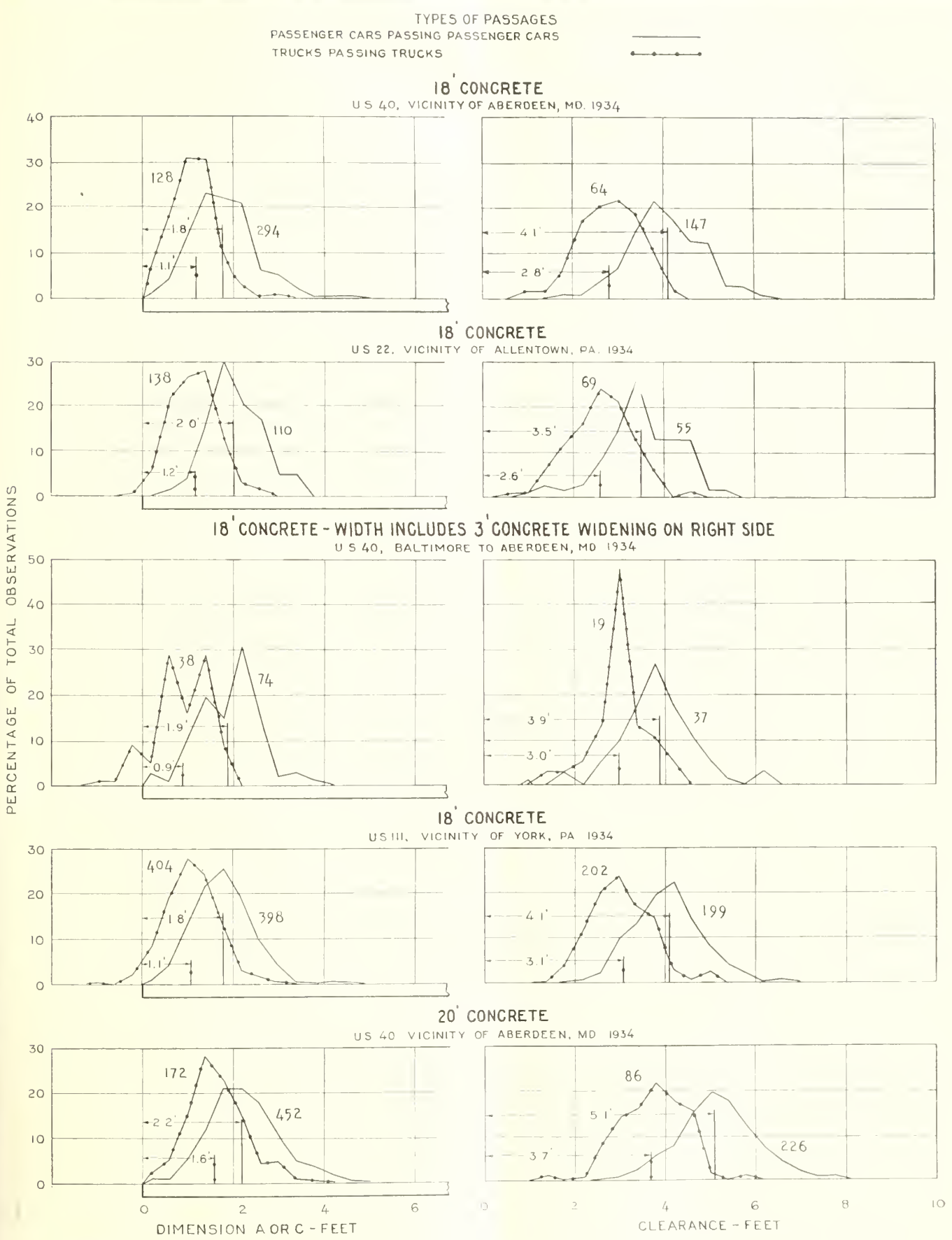

FIGURE 10.—FREQUENCY DISTRIBUTION OF EDGE DISTANCES AND CLEARANCES FOR OPPOSITE DIRECTION PASSING.

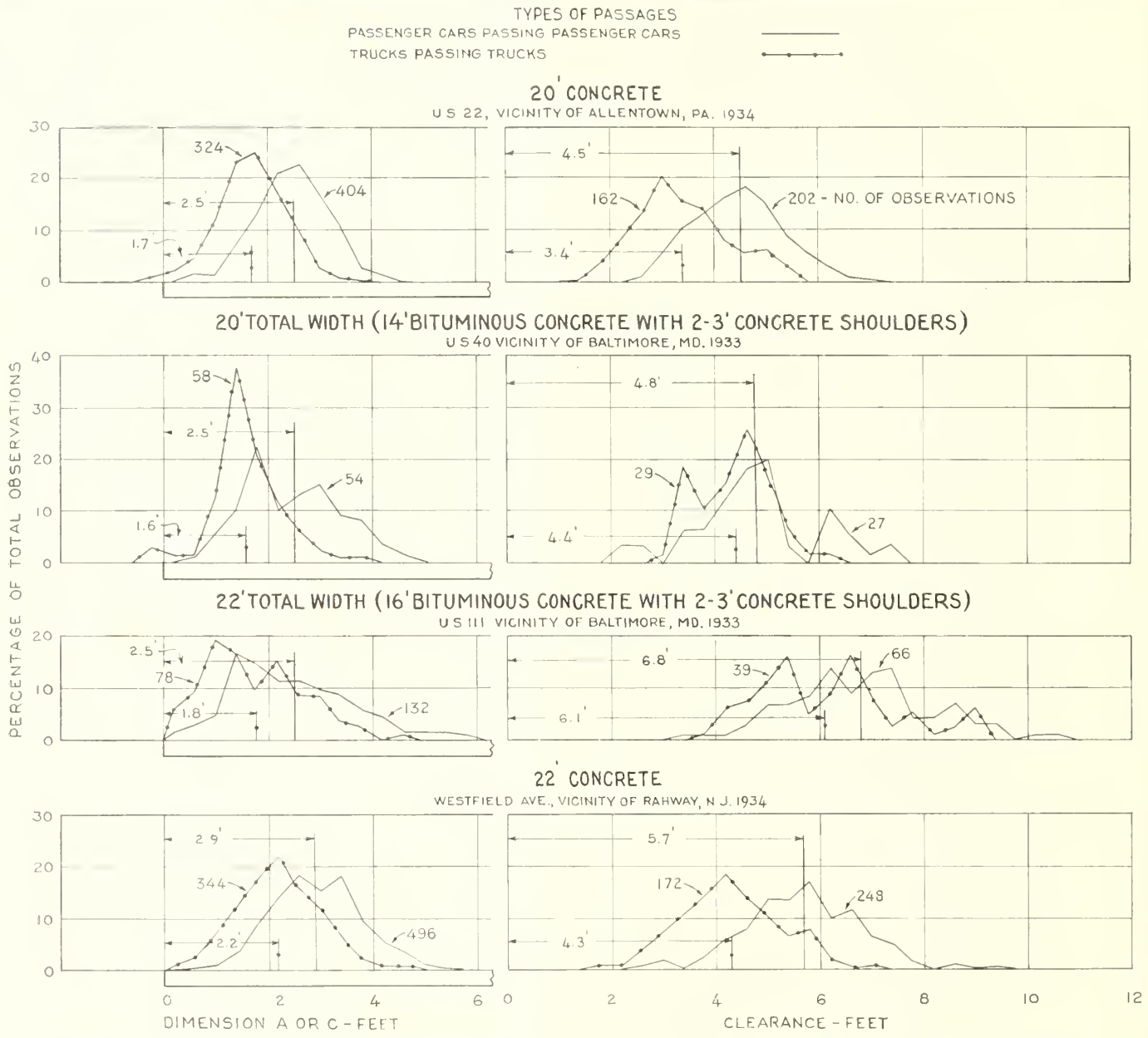

FIGURE 11.—FREQUENCY DISTRIBUTION OF EDGE DISTANCES AND CLEARANCES FOR OPPOSITE DIRECTION PASSING.

the narrow 18-foot road, 0.3 foot on the less restricted 20-foot road, and 0.7 foot on the relatively wide 22-foot road where greater choice is known to exist. This increase in used width of road should not be attributed particularly to trucks of large rated capacity. In the study of truck widths referred to above it was found that wide trucks are approximately evenly distributed among the rated capacity classes from 1½ tons to 5 tons. Eight feet is the common legal maximum width and of observed trucks of this width there were more 1½-ton trucks than 5-ton trucks.

Figures 14 to 19 present additional information on the influence of truck widths on the positions of passenger vehicles in passing trucks. An analysis was made by truck-width classes of the observations in which passenger cars in overtaking and passing trucks were, for any reason, within 1 foot of the left edge of the pavement or off of it entirely. The ends of the horizontal lines shown on the diagrams represent the positions of the right rear wheel of the critical truck and the left rear wheel of the passing car. The average edge distances found for the particular road width is designated, and also the average clearance. Truck widths were broken down into four classes: 6—7, 7—7.5, 7.5—8, and over 8 feet. The information collected on the 18-foot and 20-foot roads is summarized in table 6. The number of observations on 22-foot roads where passenger vehicles, in passing trucks, were within 1 foot of the left edge was negligible.

Table 6 shows that as truck widths increase the percentage of unfavorable left edge distances, as here defined, remains approximately constant. On the 20-foot road, which more nearly approaches a satisfactory width, this is particularly true throughout the range of truck widths, even for trucks exceeding the common legal limit of 8 feet. On the 18-foot road the percentage of such cases remains approximately constant until extralegal widths are reached, when there is a very sudden increase. Few trucks of extralegal width were observed and the sample is rather small to be considered a basis for definite conclusions.

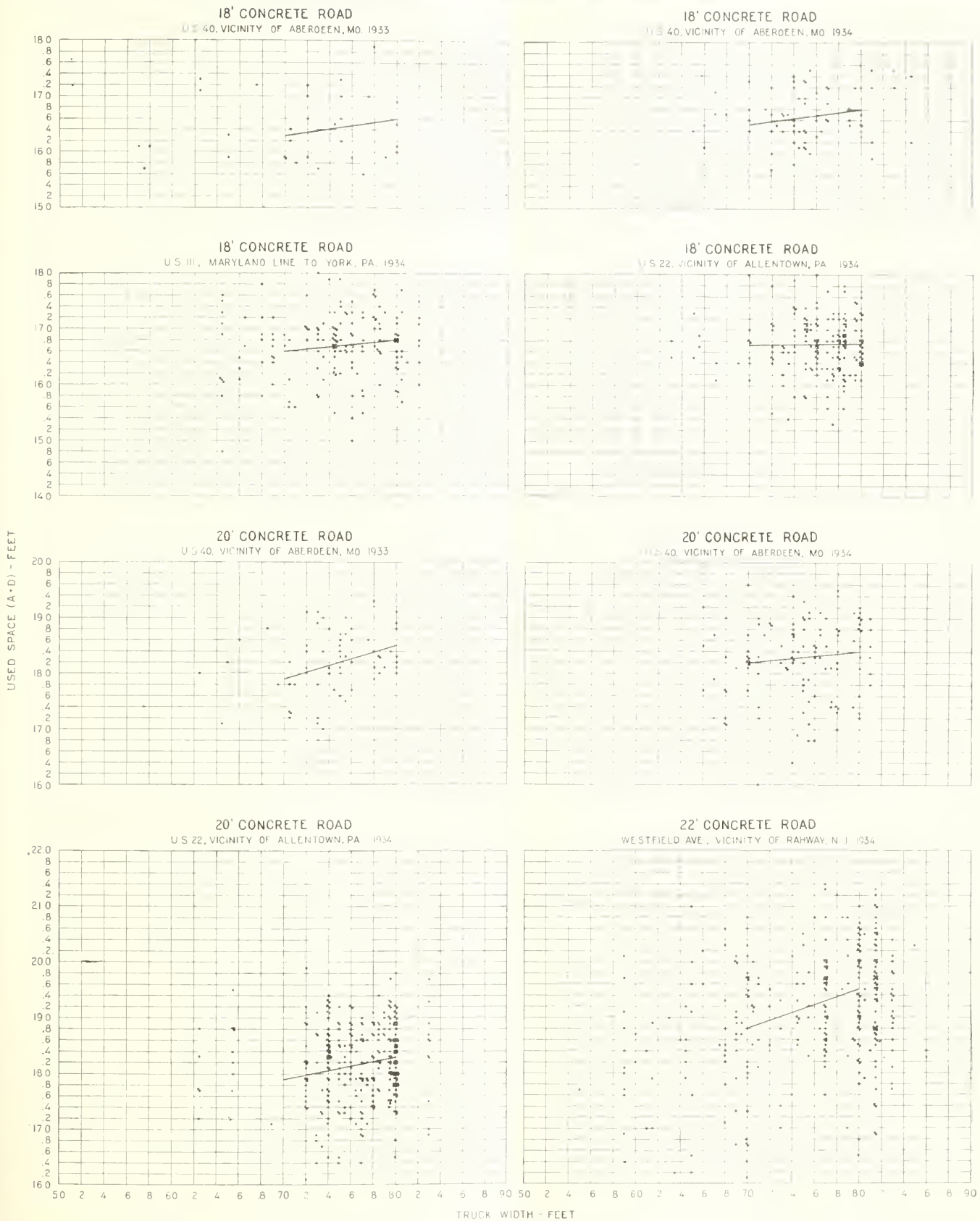
18' CONCRETE ROAD
U.S. 40, VICINITY OF ABERDEEN, MO. 1933
18' CONCRETE ROAD
U.S. 40, VICINITY OF ABERDEEN, MO. 1934
18' CONCRETE ROAD
U.S. 111, MARYLAND LINE TO YORK, PA. 1934
18' CONCRETE ROAD
U.S. 22, VICINITY OF ALLENTOWN, PA. 1934
20' CONCRETE ROAD
U.S. 40, VICINITY OF ABERDEEN, MD. 1933
20' CONCRETE ROAD
U.S. 40, VICINITY OF ABERDEEN, MD. 1934
20' CONCRETE ROAD
U.S. 22, VICINITY OF ALLENTOWN, PA. 1934
22' CONCRETE ROAD
WESTFIELD AVE., VICINITY OF RAHWAY, N.J. 1934
USED SPACE (A+D) - FEET
TRUCK WIDTH - FEET

FIGURE 12.—TRUCK WIDTHS COMPARED WITH USED SPACE FOR PASSENGER VEHICLES PASSING TRUCKS IN SAME DIRECTION. INDIVIDUAL CASES PLOTTED AND TREND LINE DETERMINED BY METHOD OF LEAST SQUARES.

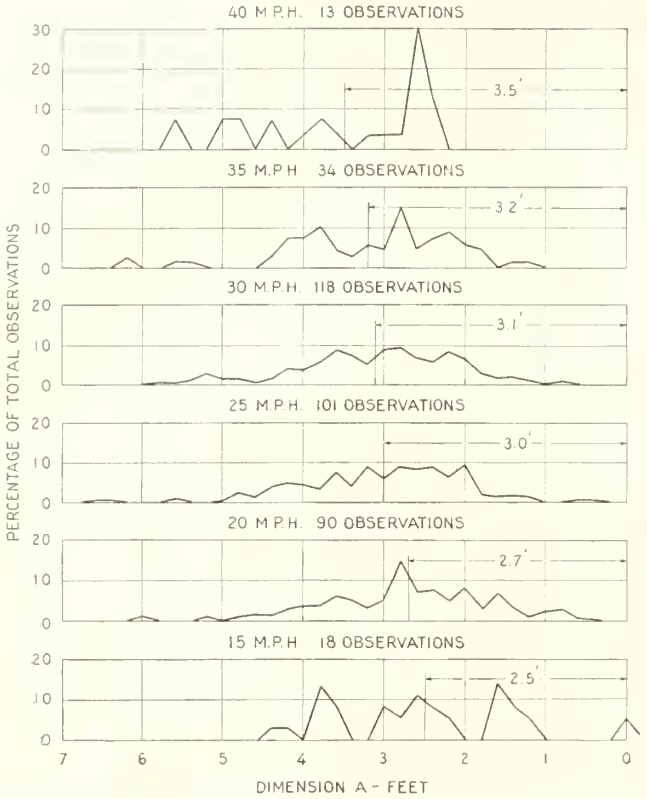

FIGURE 13.—AVERAGE POSITION OF OUTER WHEEL OF CRITICAL VEHICLE FROM RIGHT EDGE OF ROAD (DIMENSION A) AT VARIOUS SPEEDS. WESTFIELD AVENUE 22-FOOT CONCRETE ROAD.

TABLE 6.—*Results of analysis of passenger cars passing trucks of various widths where passing vehicle was within 1 foot of the left road edge or off the road* [1]

18-FOOT CONCRETE ROAD

Truck width (feet)	Number of observations of passenger cars passing trucks in each width class	Percentage of total	Observations where the passing vehicle was within 1 foot of left edge or off road	
			Number	*Percent*
6.0–7.0	67	15	18	27
7.0–7.5	138	32	47	34
7.5–8.0	213	48	66	31
Over 8.0	23	5	11	48
Total	441	100	142	32

20-FOOT CONCRETE ROAD

6.0–7.0	15	9	6	13
7.0–7.5	157	33	21	13
7.5–8.0	258	54	36	14
Over 8.0	20	4	3	15
Total	480	100	66	14

[1] Tabulation for 22-foot road omitted because of the small number of observations where the vehicle was within 1 foot of the left edge or off the road.

From the foregoing it may be argued that the width of the truck is of less importance, comparatively, than the use of excessive right edge distance, excessive clearance, or a combination of the two in causing the passing vehicle to travel close to the left edge of the pavement.

Detailed study of figures 14 to 19 shows that, in general, where less than normal clearance between vehicles was found, the passing vehicle was forced over by the selfish position taken by the passed vehicle. In

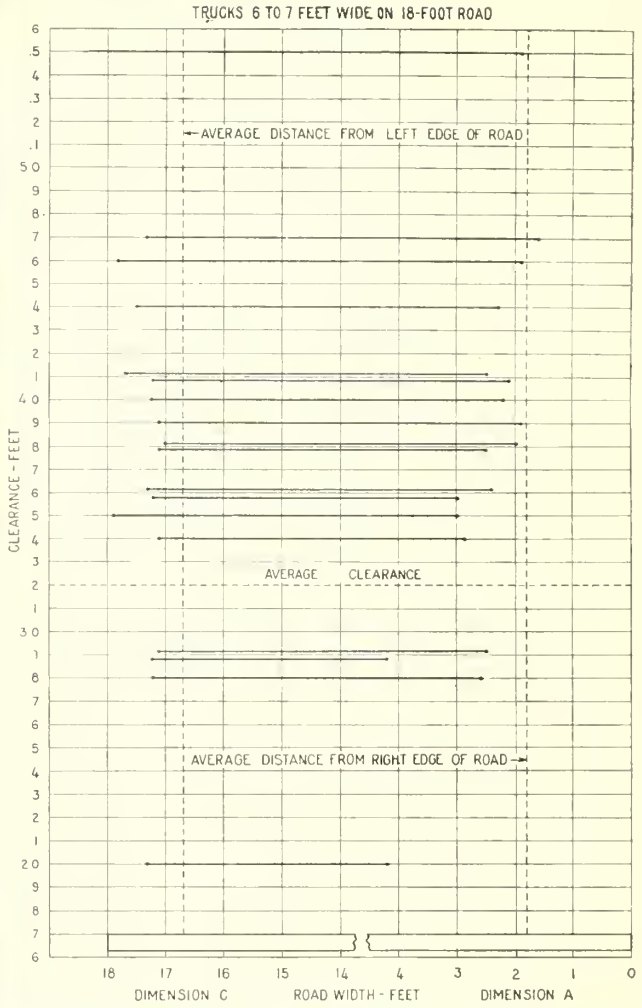
TRUCKS 6 TO 7 FEET WIDE ON 18-FOOT ROAD
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE CLEARANCE
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
CLEARANCE - FEET
DIMENSION C
ROAD WIDTH - FEET
DIMENSION A

FIGURE 14.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN 1 FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DASH LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 18-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

nearly all passings observed, where the clearance between vehicles was less than the average, the critical vehicle was taking more than the average edge distance. However, examination of those passings where the passing vehicle was close to the left edge shows more cases where the average clearance between vehicles was exceeded than there were below the average clearance. This suggests that about as many drivers run close to the left edge or off the road because of their own driving habits as are forced to by drivers of passed vehicles.

A comparison of the road width used in passing (A+D) in table 4 shows that passenger cars passing passenger cars used 0.8 foot less space than trucks passing trucks on each of the three road widths. Passages involving trucks and passenger cars required an intermediate amount of space.

The last column in table 4 shows the used space expressed as a percentage of road width. As the road width increases there is, for each vehicle class, a decrease of about 2 percent between the 18- and the 20-

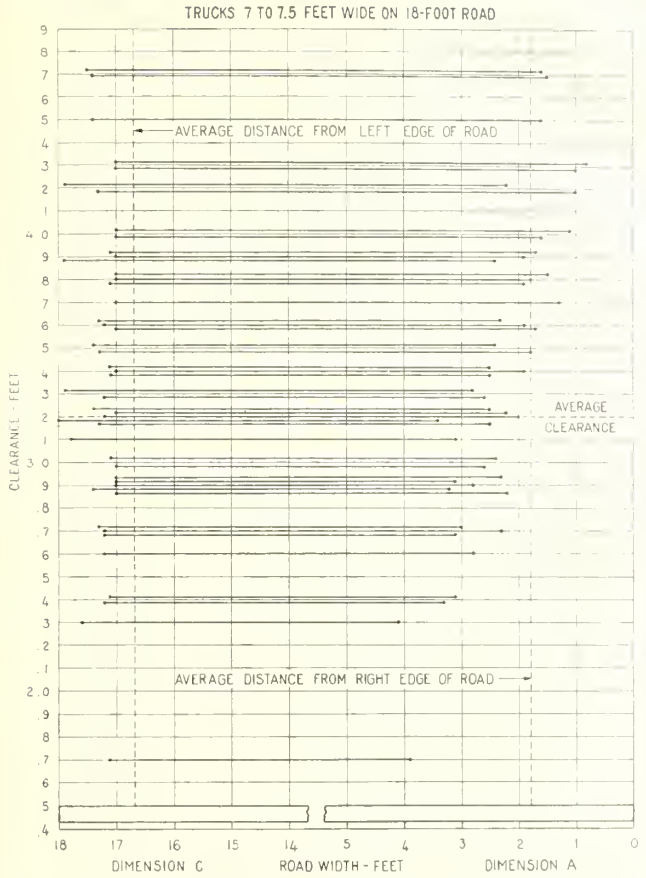
TRUCKS 7 TO 7.5 FEET WIDE ON 18-FOOT ROAD
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE CLEARANCE
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
CLEARANCE - FEET
DIMENSION C
ROAD WIDTH - FEET
DIMENSION A

FIGURE 15.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN 1 FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DOTTED LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 18-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

foot width followed by a much larger decrease between the 20- and 22-foot widths. This is again indicative of narrowness in the 18- and 20-foot roads and also of the release from width restriction that is experienced when a width of 22 feet is reached.

Figure 20 shows passenger cars passing on a 20-foot road.

CONCLUSIONS

1. Drivers of critical vehicles when being overtaken and passed tend to follow the centerline of their own traffic lane very closely.

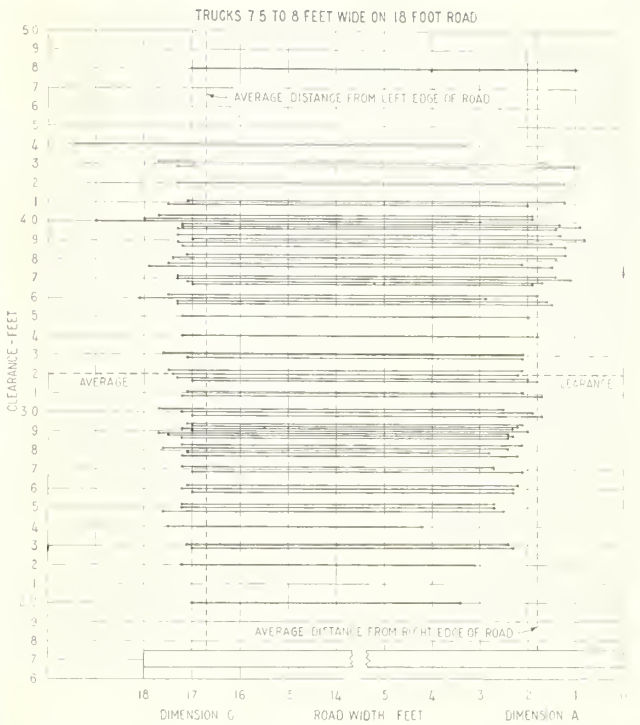
TRUCKS 7.5 TO 8 FEET WIDE ON 18 FOOT ROAD
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE
CLEARANCE
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
CLEARANCE - FEET
DIMENSION C
ROAD WIDTH - FEET
DIMENSION A

FIGURE 16.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN 1 FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DOTTED LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 18-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

2. Pavements of 18-foot width are too narrow for modern passenger cars alone or for modern mixed traffic. Pavements of 20-foot width are reasonably adequate for light-traffic roads used infrequently by wide trucks but are inadequate for heavy mixed traffic. Pavements of 22-foot width are entirely adequate for modern mixed traffic.

3. When passenger cars occupy unfavorable positions with respect to the left road edge in passing trucks, they do so because of the habits of the drivers as often as because of their being crowded over by the passed vehicle.

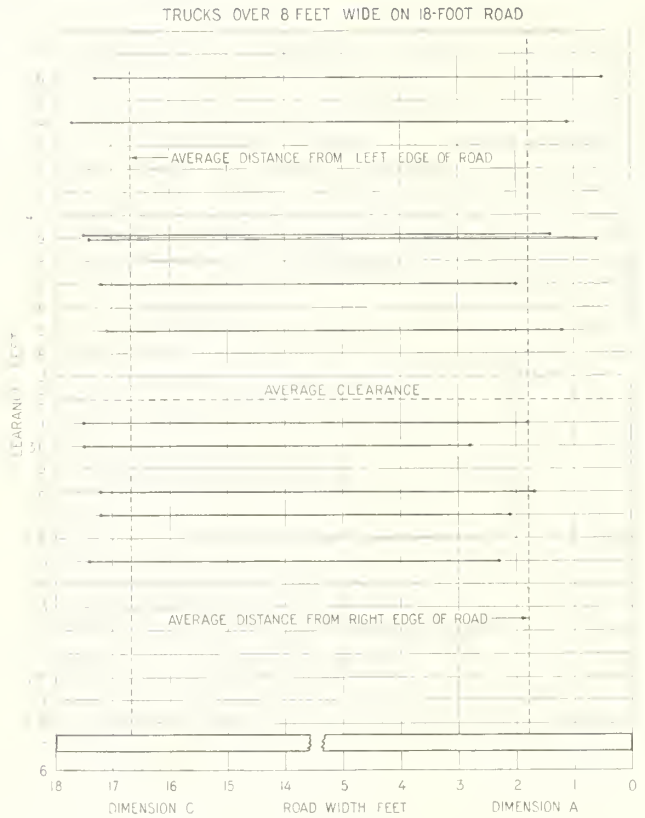
TRUCKS OVER 8 FEET WIDE ON 18-FOOT ROAD
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE CLEARANCE
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
18 17 16 15 14 5 4 3 2 1 0
DIMENSION C
ROAD WIDTH FEET
DIMENSION A

FIGURE 17.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN 1 FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DOTTED LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 18-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

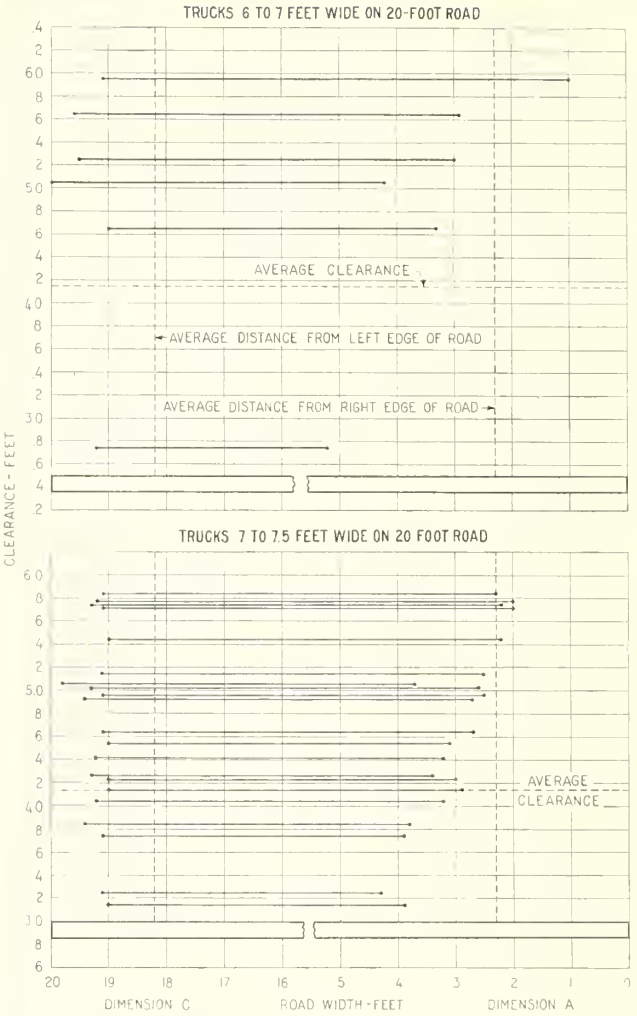
TRUCKS 6 TO 7 FEET WIDE ON 20-FOOT ROAD
AVERAGE CLEARANCE
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
CLEARANCE - FEET
TRUCKS 7 TO 7.5 FEET WIDE ON 20 FOOT ROAD
AVERAGE CLEARANCE
20 19 18 17 16 5 4 3 2 1 0
DIMENSION C
ROAD WIDTH - FEET
DIMENSION A

FIGURE 18.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN 1 FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DOTTED LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 20-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

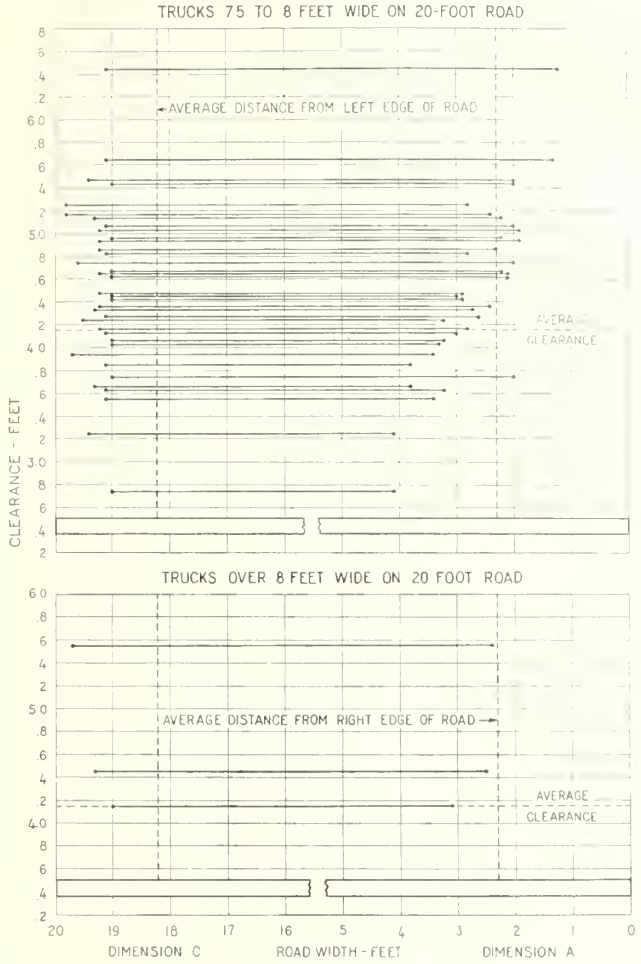
TRUCKS 7.5 TO 8 FEET WIDE ON 20-FOOT ROAD
AVERAGE DISTANCE FROM LEFT EDGE OF ROAD
AVERAGE CLEARANCE
CLEARANCE - FEET
TRUCKS OVER 8 FEET WIDE ON 20 FOOT ROAD
AVERAGE DISTANCE FROM RIGHT EDGE OF ROAD
AVERAGE CLEARANCE
20 19 18 17 16 5 4 3 2 1 0
DIMENSION C
ROAD WIDTH - FEET
DIMENSION A

FIGURE 19.—GRAPHICAL PRESENTATION OF EDGE DISTANCES AND CLEARANCE WHERE PASSENGER VEHICLES, IN OVERTAKING AND PASSING TRUCKS, PLACED THE LEFT WHEEL WITHIN ONE FOOT OF THE EDGE OF PAVEMENT. THE ENDS OF THE HORIZONTAL LINES SHOW THE POSITION OF THE RIGHT REAR WHEEL OF TRUCKS AND OF THE LEFT REAR WHEEL OF THE PASSENGER VEHICLES. THE AVERAGE DISTANCES SHOWN BY DOTTED LINES ARE FOR ALL OBSERVATIONS OF PASSENGER VEHICLES PASSING TRUCKS ON 20-FOOT PAVEMENTS REGARDLESS OF DISTANCES TO EDGES.

FIGURE 20.—PASSING OPERATIONS ON A 20-FOOT ROAD.

PUBLICATION ON HIGHWAY BRIDGES AVAILABLE

"Highway Bridge Surveys", a booklet which describes with clarity and in complete detail the importance of the various kinds of data needed in the design of bridges, is being reprinted by the Superintendent of Documents and will soon be available.

The importance of a comprehensive and accurate bridge survey can hardly be overemphasized, the booklet states. Incomplete or inaccurate information may quickly result in bridge failure, involving financial loss as well as possible loss of human life. All pertinent data for each bridge should be obtained and filed, as each structure built may be considered to constitute a practical experiment in bridge building. Such service records furnish additional data that further advance the art of bridge building.

Civil engineering instructors and students will find this publication invaluable as an exhaustive but concise textbook, complete with sample forms for recording data, illustrations, diagrams, necessary formulas, etc.

Written by Mr. C. B. McCullough, an outstanding authority on bridges, this 76-page booklet was first issued several years ago. Published as United States Department of Agriculture Technical Bulletin No. 55, "Highway Bridge Surveys" may be obtained from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 20 cents per copy. A 25-percent price reduction can be obtained on single orders for 100 or more copies.

DISPOSITION OF STATE MOTOR-FUEL TAX RECEIPTS, 1936

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed balances, etc.[1]	Net total funds distributed [2]	Expenses of collection and administration	For other administrative purposes [3]	For State highway purposes							For local roads and streets [6]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes					
						Construction, maintenance, and administration [4]	State highway police	Service of State highway obligations				Total for State highway purposes	For work on county and local roads	For work on city streets [7]	Service of local highway obligations	Total		To general funds [8]		For relief of unemployment or destitution	For education	For other purposes	Total
								State highway bonds	State-assumed local obligations [5]	Notes and other short-term loans	Total							Inspection fees, dealers' licenses, etc.[9]	Motor-fuel tax				
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	11,803	−78	11,725	51	49	4,374		1,508			1,508	5,882	5,743			5,743							
Arizona	3,843		3,843	63		2,533	91					2,624	*1,149			1,149						[10] 7	7
Arkansas	9,235	−23	9,212	341	3	2,292		3,536	2,249	58	5,843	8,135	572		84	656		77					77
California	43,008	330	43,338	139		28,547						28,547	*10,713	3,939		14,652							
Colorado	6,833	−10	6,823	99	4	4,733	179					4,912	[11] 1,808			1,808							
Connecticut	8,835	−182	8,653	51		7,930			672		672	8,602											
Delaware	1,856		1,856	12		1,035	119	45	325		370	1,524	[12] 320			320							
Florida	20,317	−15	20,302	19	358	8,360	146		2,846		2,846	11,352	12		5,693	5,705			2,846			[13] 22	2,868
Georgia	17,493		17,493	522		8,853			2,403		2,403	11,256	2,814			2,814				2,901			2,901
Idaho	3,696	−12	3,684	13		3,671						3,671											
Illinois	33,819	7	33,826	175	261	11,097						11,097	*7,658	7,422		15,080		99		4,417	2,697		7,213
Indiana	21,154	−2,857	18,297	84	70	7,449						7,449	8,245	2,061		10,306		388					388
Iowa	12,196	5	12,201	78	23	3,470			3,250		3,250	6,720	*5,380			5,380							
Kansas	9,520	−61	9,459	314	102	5,898	12		681		681	6,591	2,452			2,452							
Kentucky	11,277	253	11,530	35		11,402	49					11,451	41			44							
Louisiana	12,207	−939	11,268	94				7,687			7,687	7,687								1,576	956	[14] 955	3,487
Maine	5,202	57	5,259	16		3,172	150	1,411			1,411	4,733	510			510							
Maryland	8,921		8,921	34		3,107		1,534			1,534	4,641	993	2,667	572	4,232						[15] 14	14
Massachusetts	18,448	15	18,463	50		7,414	255	1,319			1,319	8,988	2,879		353	3,232	[16] 751		3,650	[17] 1,792			5,442
Michigan	25,739	−141	25,598	141		15,182		3,720			3,720	18,902	*6,550			6,550		5					5
Minnesota	12,329	−85	12,244	113		8,435	192					8,627	*3,421			3,421		83					83
Mississippi	9,062	−228	8,834	75	56	2,905	30	1,761		236	1,997	4,932	*3,753			3,753	18						
Missouri	11,188		11,188	55	50	6,794	148	4,075			4,075	11,017						66					66
Montana	4,455	−149	4,306	22		4,150		134			134	4,284											
Nebraska	11,320	−30	11,290	102		5,594						5,594	*3,022	334		3,356				2,238			2,238
Nevada	1,080		1,080	3		1,041	19		14		14	1,077											
New Hampshire	3,181	−1	3,180	[18]		2,105		800			800	2,905	133		[19] 142	275							
New Jersey	19,106	1,432	20,538	121		2,051		7,636			7,636	9,687	3,120		345	3,465	16			5,388	1,285	[20] 576	7,249
New Mexico	3,410	−25	3,385	75		1,746		1,564			1,564	3,310											
New York [21]	55,709	1,060	56,769	91		5,544	599	3,874			3,874	10,017	*8,104			8,104			[22] 38,557				38,557
North Carolina	20,961		20,961	[23]	30	4,972	203	6,641	407		7,048	12,223	[12] 6,672			6,672		932	1,104				2,036
North Dakota	2,301	30	2,331	25	56	1,476	24					1,500	750			750							
Ohio	[24] 43,450	203	43,653	188		18,062						18,062	7,882	[25] 6,376		14,258					11,145		11,145
Oklahoma	13,216	−62	13,154	264		5,777						5,777	*3,231			3,231		5				[26] 3,877	3,882
Oregon	9,218	−69	9,149	34		4,523	203	2,976			2,976	7,702	*1,179			1,179	59		175				175
Pennsylvania	49,383	[27] −4,646	44,737	257		21,147	862	2,807			2,807	24,816	*6,021	1,192		7,213	62			12,219		[28] 170	12,389
Rhode Island	2,230	1	2,231	14		1,095	77	132			132	1,304					13		607	293			900
South Carolina	9,695	−9	9,686	[29] 41	[29] 12	1,902		1,037	4,968		6,005	7,907	*1,579			1,579		147					147
South Dakota	4,186	−3	4,183	47	118	1,833						1,833					54					[30] 2,131	2,131
Tennessee	18,158	−93	18,065	181	93	2,046		4,928	2,372		7,300	9,346	4,842			4,842		938				[31] 2,665	3,603
Texas	38,471	−691	37,780	552		18,614			9,307		9,307	27,921									9,307		9,307
Utah	3,088	−26	3,062	7	4	2,908	143					3,051											
Vermont	2,277	−117	2,160	3		1,159		333			333	1,492	665			665							
Virginia	14,714		14,714	[23]	29	7,972			552		552	8,524	[32] 6,157			6,157						[33] 4	4
Washington	14,345	2	14,347	24		3,722		[34] 112			112	3,834	*7,812	1,644	[34] 91	9,547				[34] 912			942
West Virginia	6,810		6,810	12		1,769		5,029			5,029	6,798											
Wisconsin	18,028	−1,156	16,872	63	197	6,784			2,382		2,382	9,166	4,465	753		5,218	86		[35] 2,142				2,142
Wyoming	2,254	−32	2,222	12		1,507	36	112			112	1,655	555			555							
District of Columbia	2,393	−1	2,392	[36]										2,382		2,382		10					10
Total	691,420	−8,346	683,074	4,712	1,515	288,155	3,537	64,711	32,428	294	97,433	389,125	131,205	28,770	7,280	167,255	1,059	2,750	49,081	28,865	28,291	10,421	119,408

[1] Amounts distributed during the calendar year differ in many cases from actual collections because of undistributed balances and lag between accounts of collecting and expending agencies. Adjustments also include deduction of proceeds of tax on gasoline used in aviation in Idaho, Michigan, Nebraska, Oregon, South Carolina, and Wyoming.

[2] In many States the proceeds of motor-fuel taxes, motor-vehicle fees, and motor-carrier taxes are placed in a common fund from which the distribution is made. In these cases the amounts distributed have been prorated in proportion to the receipts, not otherwise dedicated, from these 3 sources of revenue. See tables pages 140 to 143.

[3] Where reported separately from collection expenses, funds allotted for motor-fuel inspection, administration of motor vehicle department, and regulation of motor vehicles, are shown in this column.

[4] Includes funds allotted for expenditure on urban extensions of State highway system, where reported separately from other funds distributed for local roads and streets.

[5] County or local obligations assumed by State as reimbursement for local roads added to State system.

[6] In States indicated by star (*) law provides that allotments for work on local roads or streets may also be used for service of local highway obligations, but amounts so used not reported separately.

[7] In a number of States, allotments for local road work may be used on city streets. This column shows allotments which were reported separately. See note 4.

[8] To State general funds unless otherwise noted. Allocations to county or municipal general funds may have been used in part for highways, but such amounts not reported.

[9] As fees for inspection of gasoline, dealers' license fees, and penalties for infractions of the motor-fuel tax law are not ordinarily regarded as highway revenues, the allocation of such funds to general revenue is shown separately from the allocation of regular motor-fuel tax receipts.

[10] For engineering expenses in connection with irrigation.

[11] Funds allotted to counties for use on both State and local roads.

[12] For county roads under State control.

[13] To division of airways.

[14] For harbor improvement.

[15] To Tolchester Ferry Co.

[16] To Metropolitan District Commission.

[17] Service of nonhighway portion of Emergency Public Works loan, $1,271,000; flood relief and other expenditures for relief, $521,000.

[18] Paid out of motor-vehicle revenue, $3,500. See table pages 140 and 141.

[19] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[20] Service of institutional construction bonds, $486,000; Department of Commerce and Navigation, $90,000.

[21] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise dedicated.

[22] To State general fund after crediting appropriations for highway purposes, $37,011,000; New York City general fund, $1,546,000.

[23] Included in cost of collecting motor-vehicle revenue. See table pages 140 and 141.

[24] Ohio imposes a 3-cent tax on motor-vehicle fuel and a 1-cent tax on all liquid fuels. The receipts from the 1-cent tax applicable to nonmotor-vehicle fuels (kerosene, fuel oil, etc.) were $689,000. These receipts have been eliminated from the total given, which represents a 1-cent tax on motor-vehicle fuel.

[25] In cities situated on State highways, one-sixth municipal allotment to be used on urban extensions of State system.

[26] For service of general State debt.

[27] In computing adjustment, amounts loaned to general fund for relief purposes in 1935 and 1936, and not yet repaid, have been included in the undistributed balance.

[28] For aircraft landing fields, $121,000; cooperative work other departments, $49,000.

[29] Estimated.

[30] For payments on real estate bonds.

[31] Service of general fund bonds, $2,421,000; Great Smoky Mountain Park bonds, $242,000; aviation projects, $2,000.

[32] For county roads under State control in all but 3 counties, $5,918,000; transferred to remaining 3 counties, $239,000.

[33] For aviation purposes.

[34] Debt service charges on $10,000,000 emergency relief bond issue prorated in proportion to allotments for State highways, local roads, and nonhighway purposes.

[35] To towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles.

[36] Paid out of general revenue. Amount not reported.

DISPOSITION OF STATE MOTOR-VEHICLE RECEIPTS, 1936

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed balances, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration[3]	For other administrative purposes[4]	For State highway purposes							For local roads and streets[7]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes				
						Construction, maintenance, and administration[5]	State highway police	Service of State highway obligations				Total for State highway purposes	For work on county and local roads	For work on city streets[8]	Service of local highway obligations	Total		To general funds[9]	For relief of unemployment or destitution	For education	For other purposes	Total
								State highway bonds	State-assumed local obligations[6]	Notes and other short-term loans	Total											
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	4,101	−118	3,983	419		1,240	278	1,321			1,321	2,839						[10] 725				725
Arizona	1,028	16	1,044	190		825	29					854										
Arkansas	2,829		2,829	80		750	87	1,157	736	19	1,912	2,749										
California	21,087	−1,751	19,336	2,177	913	3,283	2,159	4,052			4,052	9,494	*3,399			3,399		[10] 3,353				3,353
Colorado	2,589	−58	2,531	639	60	888	34					922	[11] 910			910						
Connecticut	5,957	211	6,168	827		1,620	325		137		137	2,082	3,259			3,259						
Delaware	1,102	39	1,141	94		587	68	25	185		210	865	[12] 182			182						
Florida	5,546		5,546	416	209															4,921		4,921
Georgia	1,302		1,302	150	49	868			235		235	1,103										
Idaho	2,175	61	2,236	60		316	48					364	*1,812			1,812						
Illinois	19,410	−130	19,280	954	316	6,045	1,421	8,929	63		8,992	16,458	1,353			1,353		199				199
Indiana	9,044	66	9,110	863		3,180	338					3,518	1,409	352		1,761		2,968				2,968
Iowa	10,793	−16	10,777	813		5,145			4,819		4,819	9,964										
Kansas	3,815		3,815	285		2,302	5		266		266	2,573	957			957						
Kentucky	4,591	24	4,615	411	24	2,807	17					2,824	557			557		799				799
Louisiana	4,112	125	4,237	129		2,797	404	687		220	907	4,108										
Maine	3,582	44	3,626	117	9	2,118	100	942			942	3,160	340			340						
Maryland	4,744	−2,311	2,433	345	42	342	322	584			584	1,248		274		274			524			524
Massachusetts	6,795	217	7,012	1,489	35	2,756	95	490			490	3,341	1,071		131	1,202	[13] 279		[14] 666			666
Michigan	19,737	−406	19,331	1,067			250					250	*17,677			17,677		337				337
Minnesota	8,189	41	8,230	406	1,802	1,818	41	2,228	1,824		4,052	5,911						111				111
Mississippi	1,869		1,869	96		187				16	16	203	*1,570			1,570						
Missouri	8,988	42	9,030	500		5,261	114	3,155			3,155	8,530										
Montana	1,730	−96	1,634	74			102					102	1,427	31		1,458						
Nebraska	2,158	32	2,190	86		618	37					655	1,449			1,449						
Nevada	279		279	18		168	3	90			90	261										
New Hampshire	2,635	14	2,649	140	3	2,080	150	4			4	2,234	132		[15] 140	272						
New Jersey	17,851	4,600	22,451	1,107		7,575						7,575	6,445		1,273	7,718	61		5,990			5,990
New Mexico	1,318	94	1,412	213		539						539	180			180		[16] 480				480
New York[17]	46,291	−347	45,944	2,473	93	7,173	775	5,012			5,012	12,960	*9,185			9,185		[18] 21,233				21,233
North Carolina	7,589	−209	7,380	398		1,736	71	2,318	142		2,460	4,267	[12] 2,329			2,329		386				386
North Dakota	1,456	359	1,815	95	20	827	21					848	847			847				[19] 5		5
Ohio	23,256	862	24,118	807		5,359	560					5,919	*17,067			17,067				[20] 325		325
Oklahoma	4,743	60	4,803	713	305	1,550						1,550	*2,235			2,235						
Oregon	2,832	−37	2,795	331		1,222	55	804			804	2,081	*319			319	16	48				48
Pennsylvania	35,331	[21] −87	35,244	1,618		27,099	1,105	3,597			3,537	31,801		1,527		1,527	80				[22] 218	218
Rhode Island	2,596	372	2,968	271		1,332	93	161			161	1,586					16	738	357			1,095
South Carolina	1,876	57	1,933	216		376	155	205	981		1,186	1,717										
South Dakota	1,540	18	1,558	61	27	304						304	1,157			1,157	9					
Tennessee	3,706	1	3,707	195		3,236	209	67			67	3,512										
Texas	17,725	−1	17,724	937	305	5,273	316					5,589	*10,893			10,893						
Utah	976	−48	928	135	11			737			737	737						45				45
Vermont	2,245	−108	2,137	58		1,071	86	307			307	1,464	615			615						
Virginia	5,737	17	5,754	401		4,814	205		334		334	5,353										
Washington	2,980	139	3,119	318		2,658	120					2,778	23			23						
West Virginia	5,832		5,832	189		1,249	24	3,551			3,551	4,824	[12] 819			819						
Wisconsin	12,213	35	12,246	727	164	4,637			1,628		1,628	6,265	3,052	514		3,566	59	[23] 1,465				1,465
Wyoming	541	14	555	12		369	8	166			166	543										
District of Columbia	963	−109	854	95	49													[24] 710				710
Total	359,784	1,726	361,510	24,215	4,436	126,400	10,230	40,589	11,350	255	52,194	188,824	92,670	2,698	1,544	96,912	520	33,597	7,537	4,921	548	46,603

[1] Amounts distributed during the calendar year differ in many cases from actual collections because of undistributed balances and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of motor-fuel taxes, motor-vehicle fees, and motor-carrier taxes are placed in a common fund from which the distribution is made. In these cases the amounts distributed have been prorated in proportion to the receipts, not otherwise dedicated, from these 3 sources of revenue. See tables pp. 138–139 and 142–143.

[3] Collection expenses in many States include service charges deducted by county and local collectors.

[4] Where reported separately from collection expenses, funds allotted for collection of motor-fuel tax, payments to auto-theft fund, and miscellaneous expenses of motor-vehicle regulation, are shown in this column.

[5] Includes funds allotted for expenditure on urban extensions of State highway system, where reported separately from other funds distributed for local roads and streets.

[6] County or local obligations assumed by State as reimbursement for local roads added to State system.

[7] In States indicated by star (*) law provides that allotments for work on local roads or streets may also be used for service of local highway obligations, but amounts so used not reported separately.

[8] In a number of States allotments for local road work may be used on city streets. This column shows allotments which were reported separately. See note 5.

[9] To State general funds unless otherwise noted. Allocations to county or municipal general funds may have been used in part for highways, but such amounts not reported.

[10] To county and municipal general funds.

[11] Funds allotted to counties for use on both State and local roads.

[12] For county roads under State control.

[13] To metropolitan district commission.

[14] Service of nonhighway portion of Emergency Public Works loan, $473,000; flood relief and other expenditures for relief, $193,000.

[15] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[16] To State general fund, $180,000; county general funds, $300,000.

[17] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise dedicated.

[18] To State general fund after crediting appropriations for highway purposes, $16,788,000; New York City general fund, $4,445,000.

[19] To Bureau of Criminal Identification.

[20] Hospitalization of indigent persons injured in motor-vehicle accidents.

[21] In computing adjustment, amounts loaned to general fund for relief purposes in 1935 and 1936, and not yet repaid, have been included in the undistributed balances.

[22] For aircraft landing fields, $156,000; cooperative work other departments, $62,000.

[23] To towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles.

[24] To District of Columbia general fund.

DISPOSITION OF STATE MOTOR-CARRIER TAX RECEIPTS, 1936

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed balances, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration	For State highway purposes						For local roads and streets[5]				For nonhighway purposes			
					Construction, maintenance, and administration[3]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads	Service of local highway obligations	Total	For other highway purposes (park and forest roads, etc.)	To general funds[6]	For relief of unemployment or destitution	For education	Total
							State highway bonds	State-assumed local obligations[4]	Total									
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	187	−17	170	42	128					128								
Arizona	150		150	9	136	5				141								
Arkansas	3		3		1		1	1	2	3								
California	2,742	−90	2,652	564	7					7	7		7		2,074			2,074
Colorado	568	−59	509	88	251	10				261	[7] 160		160					
Connecticut	290	−107	183		55			5	5	60					[8] 123			123
Delaware	(9)																	
Florida	263		263	46								204	204		[8] 8		5	13
Georgia	400	−165	235	135	79			21	21	100								
Idaho	122	−8	114	39	49	26				75								
Illinois	(9)																	
Indiana	641	−73	568	132	436					436								
Iowa	500		500	152							*348		348					
Kansas	1,079		1,079	268	460	107		53	53	620	191		191					
Kentucky	309	−41	268	39	227	1				228	1		1					
Louisiana	4		4	4														
Maine	23		23	23														
Maryland	(10)																	
Massachusetts	84		84	66											18			18
Michigan	481	−119	362	106	256					256								
Minnesota	19		19	19														
Mississippi	100	5	105	2							*103		103					
Missouri	551	−470	81	81														
Montana	37	−4	33	33														
Nebraska	(9)																	
Nevada	199	1	200	12	185	3				188								
New Hampshire	3		3	3														
New Jersey	79	36	115		46					46	25	7	32	1		36		36
New Mexico	149	3	152	11	128	13				141								
New York	(9)																	
North Carolina	175		175		43	2	58	4	62	107	[11] 58		58		10			10
North Dakota	49	−13	36	36														
Ohio	728	−344	384	128	133					133	123		123					
Oklahoma	1,004	−21	983	49	934					934								
Oregon	1,012	−37	975	173	392	30	258		258	680	*102		102	5	15			15
Pennsylvania	5		5		5					5								
Rhode Island	11		11	10	1					1								
South Carolina	126	−32	94	22	65					65					[12] 7			7
South Dakota	479	14	493	48	433					433				12				
Tennessee	323	−5	318	56	202		9		9	211					51			51
Texas	82		82	72	10					10								
Utah	205	−109	96	21	75					75								
Vermont	(9)																	
Virginia	155		155	22	100			7	7	107					26			26
Washington	197		197	197														
West Virginia	72		72		19		53		53	72								
Wisconsin	1,144	−70	1,074	326											748			748
Wyoming	174		174	30	141	3				144								
District of Columbia	213	−1	212												[13] 212			212
Total	15,137	−1,726	13,411	3,064	4,997	200	379	91	470	5,667	1,118	211	1,329	18	3,292	36	5	3,333

[1] Amounts distributed during the calendar year differ in many cases from actual collections because of undistributed balances and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of motor-fuel taxes, motor-vehicle fees, and motor-carrier taxes are placed in a common fund from which the distribution is made. In these cases the amounts distributed bave been prorated in proportion to the receipts, not otherwise dedicated, from these 3 sources of revenue. See tables pp. 138 to 141.

[3] Includes funds allotted for expenditure on urban extensions of State highway system, where reported separately from other funds distributed for local roads and streets.

[4] County or local obligations assumed by State as reimbursement for local roads added to State system.

[5] In States indicated by star (*) law provides tbat allotments for work on local roads or streets may also be used for service of local highway obligations, but amounts so used not reported separately.

[6] To State general funds unless otherwise noted. Allocations to county or municipal general funds may have been used in part for highways, but such amounts not reported.

[7] Funds allotted to counties for use on both State and local roads.

[8] To cities and towns.

[9] No special taxes on motor carriers reported.

[10] Ton-mile and passenger-mile taxes paid by motor carriers in lieu of registration fees included in motor-vehicle receipts, table pp. 140 and 141.

[11] For county roads under State control.

[12] To cities.

[13] To District of Columbia general fund.

DISPOSITION OF RECEIPTS FROM STATE IMPOSTS ON HIGHWAY USERS, 1936

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year [1]	Adjustments due to undistributed balances, etc.[2]	Net total funds distributed	Expenses of collection and administration [3]	For State highway purposes						For local roads and streets [6]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes					
					Construction, maintenance, and administration [4]	State highway police	Service of State highway obligations				Total for State highway purposes	For work on county and local roads	For work on city streets [7]	Service of local highway obligations	Total		To general funds [8]		For relief of unemployment or destitution	For education	For other purposes	Total
							State highway bonds	State assumed local obligations [5]	Notes and other short-term loans	Total							Motor-fuel inspection fees, dealers' licenses, etc.[9]	All other highway user imposts				
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	16,091	−213	15,878	561	5,742	278	2,829			2,829	8,849	5,743			5,743			[10] 725				725
Arizona	5,021	16	5,037	262	3,494	125					3,619	*1,149			1,149						[11] 7	7
Arkansas	12,067	−23	12,044	424	3,043	87	4,694	2,986	77	7,757	10,887	572		84	656		77					77
California	66,837	−1,511	65,326	3,793	31,837	2,159	4,052			4,052	38,048	*14,119	3,939		18,058			[12] 5,427				5,427
Colorado	9,990	−127	9,863	890	5,872	223					6,095	[13] 2,878			2,878							
Connecticut	15,082	−78	15,004	878	9,605	325		814		814	10,744	3,259			3,259			[14] 123				123
Delaware	2,958	39	2,997	106	1,622	187	70	510		580	2,389	[15] 502			502							
Florida	26,126	−15	26,111	1,048	8,360	146		2,846		2,846	11,352	12		5,897	5,909			[16] 2,854		4,926	[17] 22	7,802
Georgia	19,195	**−165**	19,030	856	9,800			2,659		2,659	12,459	2,814			2,814					2,901		2,901
Idaho	5,993	41	6,034	112	4,036	74					4,110	*1,812			1,812							
Illinois	53,229	−123	53,106	1,706	17,142	1,421	8,929	63		8,992	27,555	*9,011	*7,422		16,433		99	199	4,417	2,697		7,412
Indiana	30,839	−2,864	27,975	1,149	11,065	338					11,403	9,654	2,413		12,067		388	2,968				3,356
Iowa	23,489	−11	23,478	1,066	8,615			8,069		8,069	16,684	*5,728			5,728							
Kansas	14,414	−61	14,353	969	8,660	124		1,000		1,000	9,784	3,600			3,600							
Kentucky	16,177	236	16,413	509	14,436	67					14,503	602			602			799				799
Louisiana	16,323	−814	15,509	227	2,797	404	8,374		220	8,594	11,795								1,576	956	[18] 955	3,487
Maine	8,807	101	8,908	165	5,290	250	2,353			2,353	7,893	850			850							
Maryland	13,665	−2,311	11,354	421	3,449	322	2,118			2,118	5,889	993	2,941	572	4,506			524			[19] 14	538
Massachusetts	25,327	232	25,559	1,640	10,170	350	1,809			1,809	12,329	3,950		484	4,434	[20] 1,030		3,668	[21] 2,458			6,126
Michigan	45,957	−666	45,291	1,314	15,438	250	3,720			3,720	19,408	*24,227			24,227		5	337				342
Minnesota	20,537	−44	20,493	2,340	10,253	233	2,228	1,824		4,052	14,538	*3,421			3,421		83	111				194
Mississippi	11,031	−223	10,808	229	3,092	30	1,761		252	2,013	5,135	*5,426			5,426	18						
Missouri	20,727	−428	20,299	686	12,055	262	7,230			7,230	19,547						66					66
Montana	6,222	−249	5,973	129	4,150	102	134			134	4,386	1,427	31		1,458							
Nebraska	13,478	2	13,480	188	6,212	37					6,249	*4,471	334		4,805				2,238			2,238
Nevada	1,558	1	1,559	33	1,397	25	90	14		104	1,526											
New Hampshire	5,819	13	5,832	146	4,185	150	804			804	5,139	265		[22] 282	547							
New Jersey	37,036	6,068	43,104	1,228	9,672		7,636			7,636	17,308	9,590		1,625	11,215	78			11,414	1,285	[23] 576	13,275
New Mexico	4,877	72	4,949	299	2,413	13	1,564			1,564	3,990	180			180			[24] 480				480
New York [25]	102,000	713	102,713	2,657	12,717	1,374	8,886			8,886	22,977	*17,289			17,289			[26] 59,790				59,790
North Carolina	28,725	−209	28,516	428	6,751	276	9,017	553		9,570	16,597	[15] 9,059			9,059		932	1,500				2,432
North Dakota	3,806	376	4,182	232	2,303	45					2,348	1,597			1,597						[27] 5	5
Ohio	67,434	721	68,155	1,123	23,554	560					24,114	*25,072	6,376		31,448					11,145	[28] 325	11,470
Oklahoma	18,963	−23	18,940	1,331	8,261						8,261	*5,466			5,466		5				[29] 3,877	3,882
Oregon	13,062	−143	12,919	538	6,137	288	4,038			4,038	10,463	*1,600			1,600	80		238				238
Pennsylvania	84,719	[30] −4,733	79,986	1,875	48,251	1,967	6,404			6,404	56,622	*6,021	2,719		8,740	142			12,219		[31] 388	12,607
Rhode Island	4,837	373	5,210	295	2,428	170	293			293	2,891					29		[32] 1,345	650			1,995
South Carolina	11,697	16	11,713	291	2,343	155	1,242	5,949		7,191	9,689	*1,579			1,579		147	7				154
South Dakota	6,205	29	6,234	301	2,570						2,570	1,157			1,157	75					[33] 2,131	2,131
Tennessee	22,187	−97	22,090	525	5,484	209	5,004	2,372		7,376	13,069	4,842			4,842		938	51			[34] 2,665	3,654
Texas	56,278	−692	55,586	1,866	23,897	316		9,307		9,307	33,520	*10,893			10,893					9,307		9,307
Utah	4,269	−183	4,086	178	2,983	143	737			737	3,863							45				45
Vermont	4,522	−225	4,297	61	2,230	86	640			640	2,956	1,280			1,280							
Virginia	20,606	17	20,623	452	12,886	205		893		893	13,984	[35] 6,157			6,157			26			[36] 4	30
Washington	17,522	141	17,663	539	6,380	120	[37] 112			112	6,612	*7,835	1,644	[37] 91	9,570				[37] 942			942
West Virginia	12,714		12,714	201	3,037	24	8,633			8,633	11,694	[15] 819			819							
Wisconsin	31,385	−1,193	30,192	1,477	11,421			4,010		4,010	15,431	7,517	1,267		8,784	145		[38] 4,355				4,355
Wyoming	2,969	−18	2,951	54	2,017	47	278			278	2,342	555			555							
District of Columbia	3,569	−111	3,458	144									2,382		2,382		10	[39] 922				932
Total	1,066,341	−8,346	1,057,995	37,942	419,552	13,967	105,679	43,869	549	150,097	583,616	224,993	31,468	9,035	265,496	1,597	2,750	85,970	36,438	33,217	10,969	169,344

[1] Includes receipts from (1) motor-fuel taxes, (2) motor-vehicle fees and fines, and (3) special imposts on motor vehicles operated for hire (motor-carrier taxes). See tables, pp. 138 to 143, which give distribution of these 3 classes of receipts separately.

[2] Amounts distributed during the calendar year differ in many cases from actual collections because of undistributed balances and lag between accounts of collecting and expending agencies. Adjustments also include deduction of proceeds of tax on gasoline used in aviation in Idaho, Michigan, Nebraska, Oregon, South Carolina, and Wyoming.

[3] Includes expenses of collection and administration of motor-fuel tax, motor-vehicle fees, and motor-carrier taxes, and miscellaneous expenses of motor-vehicle regulation.

[4] Includes funds allotted for expenditure on urban extensions of State highway system, where reported separately from other funds distributed for local roads and streets.

[5] County or local obligations assumed by State as reimbursement for local roads added to State system.

[6] In States indicated by star (*) law provides that allotments for work on local roads or streets may also be used for service of local highway obligations, but amounts so used not reported separately.

[7] In a number of States allotments for local road work may be used on city streets. This column shows allotments which were reported separately. See note 4.

[8] To State general funds unless otherwise noted. Allocations to county or municipa lgeneral funds may have been used in part for highways, but such amounts not reported

[9] As fees for inspection of gasoline, dealers' license fees, and penalties for infractions of the motor-fuel tax law are not ordinarily regarded as highway revenues, the allocation of such funds to general revenue is shown separately from the allocation of regular motor-fuel tax receipts.

[10] To county and municipal general fuuds.

[11] For engineering expenses in connection with irrigation.

[12] To State general fund, $2,074,000; county and municipal general funds, $3,353,000.

[13] Funds alloted to counties for use on both State and local roads.

[14] To cities and towns.

[15] For county roads under State control.

[16] To State general fund, $2,846,000; municipal general funds, $8,000.

[17] To Division of Airways.

[18] For harbor improvement.

[19] To Tolchester Ferry Co.

[20] To Metropolitan District Commission.

[21] Service of nonhighway portion of emergency public works loan, $1,744,000; flood relief and other expenditures for relief, $714,000.

[22] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[23] Service of institutional construction bonds, $486,000; Department of Commerce and Navigation, $90,000.

[24] To State general fund, $180,000; county general funds, $300,000.

[25] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise dedicated.

[26] To State general fund after crediting appropriations for highway purposes, $53,799,000; New York City general fund, $5,991,000.

[27] To Bureau of Criminal Identification.

[28] Hospitalization of indigent persons injured in motor-vehicle accidents.

[29] For service of general State debt.

[30] In computing adjustment, $17,556,000 loaned to general fund for relief purposes in 1935 and 1936, and not yet repaid, has been included in the undistributed balance.

[31] For aircraft landing fields, $277,000; cooperative work, other departments, $111,000.

[32] In addition to this amount, $3,675,000, reported as the balance in the general highway fund, Dec. 31, 1935, was reported in 1936 as no longer available for highway purposes. The latter amount represents highway user revenues of prior years, shown in previous tables as allotted for highway purposes.

[33] For payments on real estate bonds.

[34] Service of general fund bonds, $2,421,000; Great Smoky Mountain Park bonds, $242,000; aviation projects, $2,000.

[35] For county roads under State control in all but 3 counties, $5,918,000; transferred to remaining 3 counties, $239,000.

[36] For aviation purposes.

[37] Debt service charges on $10,000,000 emergency relief bond issue prorated in proportion to allotments for State highways, local roads, and nonhighway purposes.

[38] To State general fund, $748,000; towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles, $3,607,000.

[39] To District of Columbia general fund.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF AUGUST 31, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 1,639,801	$ 819,900	68.2	$ 3,744,760	$ 1,872,375	167.6	$ 5,154,905
Arizona	$ 694,770	$ 357,298	35.9	1,360,811	991,685	44.4	257,877	171,071	16.3	1,764,484
Arkansas	345,172	345,172	16.7	3,473,291	3,467,581	232.0	263,266	262,938	10.7	2,322,961
California	3,754,489	1,972,143	80.7	5,974,864	3,271,380	104.4	1,409,454	760,009	20.8	2,840,923
Colorado	893,459	499,933	33.0	2,619,223	1,454,599	88.4	124,133	64,552	1.5	2,344,136
Connecticut	362,558	179,076	2.1	579,550	281,943	7.8	82,620	41,310	.2	1,508,572
Delaware				512,312	256,067	19.5	243,905	119,934	14.8	1,107,687
Florida	134,650	67,320	6.6	2,573,372	1,286,686	60.9	397,670	198,835	6.5	2,858,060
Georgia	816,859	408,430	44.0	3,889,503	1,944,736	189.7	1,938,066	969,033	77.8	5,340,635
Idaho	908,435	542,083	70.5	1,482,620	885,910	83.0	655,042	390,486	68.0	1,147,368
Illinois	1,422,380	676,715	42.8	10,333,417	5,146,417	314.4	4,383,223	2,184,900	86.4	2,892,257
Indiana	1,709,942	854,916	36.6	5,439,218	2,719,604	150.4	1,331,848	665,924	26.1	2,106,982
Iowa	795,707	387,048	25.3	6,675,643	2,976,765	201.0	3,382,662	1,554,650	100.6	1,179,284
Kansas	870,736	408,921	35.6	5,177,422	2,573,552	223.1	1,852,464	926,227	97.7	3,376,996
Kentucky	447,254	223,627	10.8	3,965,699	1,932,850	105.6	1,366,728	683,364	83.5	2,817,722
Louisiana	103,036	51,219	2.5	9,357,799	1,255,274	25.0	946,050	401,952	39.5	2,469,714
Maine	634,170	317,085	17.6	2,224,106	1,112,053	58.4	1,142,830	571,415	22.9	278,204
Maryland	180,456	90,228	2.1	1,439,650	719,792	21.7	987,818	493,909	16.8	1,790,879
Massachusetts	10,066	5,033		4,924,270	2,462,135	22.5	604,454	302,226	2.3	2,318,938
Michigan	2,755,300	1,377,650	76.7	6,873,560	3,437,280	162.8	3,787,173	1,815,361	71.0	99,201
Minnesota	1,053,966	526,953	57.8	5,336,438	2,600,130	192.8	1,324,746	447,658	50.1	2,449,506
Mississippi	264,600	132,300	9.6	3,998,590	1,999,230	202.3	1,883,000	940,850	77.6	3,560,644
Missouri	2,605,736	1,283,758	121.4	7,359,634	3,611,718	338.2	3,091,192	1,296,098	126.8	2,586,168
Montana	1,814,791	1,017,339	136.4	2,505,320	1,406,042	154.2	1,261,915	679,449	55.2	2,326,754
Nebraska	731,814	365,907	92.9	4,954,444	2,496,584	493.7	1,290,936	614,084	81.0	2,652,067
Nevada	595,172	515,578	34.9	1,988,379	1,712,184	84.8	105,983	91,865	1.1	1,049,767
New Hampshire	12,936	6,468		589,265	290,644	8.0	256,644	126,901	5.1	1,008,498
New Jersey				2,476,848	1,160,809	22.2	39,430	19,715		2,299,692
New Mexico	782,661	473,854	54.1	2,633,256	1,718,429	146.8	1,187,054	723,913	89.4	715,430
New York	2,597,774	1,281,576	40.6	19,792,281	9,277,416	329.6	2,918,310	1,432,955	46.0	1,979,275
North Carolina	1,334,139	667,070	101.0	4,696,563	2,224,582	274.0	1,670,351	761,155	69.2	3,448,894
North Dakota	543,890	543,890	109.0	790,120	769,610	126.2	486,842	486,842	65.1	3,921,891
Ohio	937,282	461,998	16.2	9,498,290	4,656,274	105.0	1,451,820	725,910	13.4	6,218,678
Oklahoma	946,041	497,135	32.1	3,367,100	1,746,498	136.6	1,301,247	683,208	75.7	3,747,112
Oregon	1,156,664	700,251	53.7	3,578,438	2,141,103	114.9	563,037	323,476	27.4	1,114,302
Pennsylvania	2,762,712	1,373,188	33.9	10,313,435	5,144,002	151.0	3,483,694	1,732,396	51.1	4,473,787
Rhode Island	275,020	137,510	2.8	1,320,196	660,098	16.5	10,890	4,134		902,980
South Carolina	1,010,145	420,200	68.0	5,236,124	2,139,558	280.0	630,496	282,935	40.3	2,014,011
South Dakota	674,302	399,826	46.8	1,745,878	970,716	191.7	1,139,948	630,660	101.1	3,546,060
Tennessee	589,818	294,909	25.7	1,634,344	817,172	50.2	935,240	467,620	32.6	4,936,328
Texas	3,628,267	1,812,886	281.6	10,609,662	5,284,686	615.4	2,595,590	1,292,855	111.2	7,933,732
Utah	119,208	85,202	6.2	1,350,350	970,163	134.1	515,269	353,310	34.1	1,178,379
Vermont	510,922	246,154	12.3	1,306,739	586,863	37.1	253,120	92,429	3.5	323,344
Virginia	556,620	278,310	29.5	3,099,526	1,510,673	96.2	997,241	498,620	42.3	2,601,559
Washington	903,288	474,700	53.8	2,802,362	1,466,326	47.1	484,901	253,043	9.6	1,290,137
West Virginia	384,604	192,302	12.5	1,562,782	781,039	42.6	595,054	372,622	16.1	2,341,316
Wisconsin	3,066,543	1,477,210	88.6	6,388,302	3,043,365	181.5	2,255,891	1,080,900	70.9	1,531,986
Wyoming	573,928	353,948	67.0	2,345,335	1,413,229	259.2	552,020	336,700	46.1	448,232
District of Columbia										
Hawaii	236,496	117,895	4.4	616,749	301,733	12.8	371,350	182,080	5.4	1,227,500
Puerto Rico										625,000
TOTALS	47,508,778	24,902,214	2,132.3	204,412,881	101,897,085	7,027.9	62,555,254	31,384,854	2,278.4	120,172,937

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF AUGUST 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,639,922	$ 3,602,858	130.1	$ 408,640	$ 408,640	8.1	$ 80,572	$ 80,572	6.5	$ 59,045
Arizona	2,569,841	3,009,710	2,418,697	188.6	144,128	73,622	7.3				77,522
Arkansas	3,352,061	2,951,502	2,932,664	324.5	380,793	379,276	35.8				40,121
California	7,747,928	7,419,903	7,179,790	253.6	721,829	554,363	10.3				13,775
Colorado	3,395,263	2,505,903	2,459,381	101.0	89,597	89,596	6.0	8,200	8,200		838,086
Connecticut	1,418,709	783,035	732,184	9.5	597,540	579,820	12.9	124,130	64,435	.2	42,270
Delaware	900,310	605,918	580,185	48.9	278,155	278,155	17.9				41,970
Florida	2,597,144	2,511,247	2,448,539	94.7	106,968	106,968	4.3				41,638
Georgia	4,988,967	1,094,740	1,076,584	73.9	1,671,783	1,671,783	94.4	1,297,474	1,297,474	51.2	943,126
Idaho	2,222,747	2,248,945	2,151,909	185.6	49,205	49,131	.3				21,707
Illinois	8,694,009	7,896,767	7,726,978	444.8	887,743	887,743	43.2				79,288
Indiana	4,941,255	4,290,541	4,072,063	180.9	920,670	862,447	57.5				6,745
Iowa	4,991,664	4,741,882	4,441,051	519.8	500,418	491,207	8.5	59,808	57,770	.3	1,636
Kansas	4,994,975	4,311,232	4,270,824	347.3	672,665	630,493	28.9	35,840	35,840	13.9	57,818
Kentucky	3,726,271	3,227,153	3,099,896	344.4	548,005	548,005	13.6				78,370
Louisiana	2,890,429	2,426,574	2,169,478	157.0	660,493	599,816	10.7	112,699	74,701	10.4	46,434
Maine	1,676,799	1,476,926	1,464,500	68.6	187,028	184,429	6.8	27,870	27,870	.7	
Maryland	1,750,738	471,481	464,705	17.6	747,571	747,571	17.4	193,134	159,302	6.4	379,160
Massachusetts	3,262,885	391,467	391,467	4.0	2,609,120	2,218,350	14.4	1,149,906	581,543	.8	71,525
Michigan	6,301,414	6,480,310	5,940,287	287.2	297,637	296,521	4.8	49,900	39,282	.3	25,324
Minnesota	5,277,145	5,898,286	4,920,831	887.3	557,634	332,378	15.3	42,575	23,936	.8	
Mississippi	3,457,552	2,659,690	2,655,872	184.3	698,700	697,660	50.4	40,000	40,000	1.5	64,020
Missouri	6,012,652	4,981,893	4,883,496	770.8	1,118,577	988,205	6.9	34,390	32,294		108,657
Montana	3,676,416	3,432,741	3,421,094	192.8	278,930	237,591	9.9	8,462	8,462		9,269
Nebraska	3,870,739	3,130,686	3,031,142	329.8	605,971	605,968	39.5	226,511	226,511	3.2	7,118
Nevada	2,243,074	2,291,081	2,209,009	110.0	33,646	33,646					419
New Hampshire	945,225	789,220	758,968	34.4	160,072	159,287	5.8				26,970
New Jersey	3,129,805	1,060,687	1,057,687	16.8	2,038,819	2,025,664	18.6	6,110	6,110		40,344
New Mexico	2,871,397	2,605,098	2,600,277	196.2	210,438	210,438	16.0	14,681	12,196		48,486
New York	11,046,377	10,350,071	9,920,008	158.4	543,989	543,989	11.8	130,000	130,000	.5	452,380
North Carolina	4,720,173	3,401,910	3,331,340	227.2	1,335,770	1,335,770	63.5	39,700	39,700	.5	13,363
North Dakota	2,867,245	2,388,075	2,362,829	362.8	197,305	197,249	13.6	292,734	292,734	39.8	14,433
Ohio	7,670,815	5,560,739	5,479,661	216.2	2,007,417	1,973,367	76.3	188,130	182,130	4.2	35,657
Oklahoma	4,580,670	4,140,688	4,042,366	386.6	492,646	492,586	18.6	77,400	35,307	3.8	10,410
Oregon	3,038,642	2,763,204	2,656,321	158.6	513,539	369,706	6.0				12,615
Pennsylvania	9,347,797	3,003,215	2,834,276	145.5	5,626,723	5,268,520	108.9	1,405,621	1,099,888	31.4	145,113
Rhode Island	989,208	1,109,360	986,896	18.8	2,240	2,240					72
South Carolina	2,702,012	2,179,123	2,059,166	220.1	650,935	587,605	29.5	21,242	21,242	1.5	33,998
South Dakota	2,976,454	2,339,531	2,336,310	426.4	614,617	614,617	78.2	9,972	9,972		15,555
Tennessee	4,192,460	2,759,284	2,732,492	114.8	1,185,681	1,185,681	32.6	123,170	123,170	1.2	151,117
Texas	11,989,350	12,609,144	11,546,037	1,106.4	384,461	285,584	5.9	159,161	143,575	13.2	14,155
Utah	2,067,154	1,952,289	1,751,363	190.6	277,275	276,714	17.5	11,021	11,021	.6	28,056
Vermont	924,306	1,016,181	883,048	21.9	49,422	36,400	1.3				4,858
Virginia	3,652,667	3,337,365	3,267,100	1,001.9	180,843	180,843	24.1	58,618	58,506	3.9	146,218
Washington	3,026,161	3,314,896	2,913,200	163.4	85,272	85,272	.9	25,465	20,845	.7	6,844
West Virginia	2,231,412	1,158,692	1,149,044	52.4	1,178,085	1,042,188	42.7	47,560	40,180	5.4	
Wisconsin	4,823,884	5,205,645	4,690,599	337.7	133,855	124,000	6.1	5,025	4,900		4,385
Wyoming	2,219,155	2,188,251	2,182,594	152.4	33,287	33,287					3,274
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	623,701	605,700	8.9	334,743	265,689	8.5				54,644
TOTALS	195,000,000	161,685,904	153,842,262	11,984.2	34,010,880	31,850,080	1,111.5	6,107,081	4,989,668	202.9	4,317,990

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF AUGUST 31, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,041,147	$ 3,040,883	42	1	12	$ 798,172	$ 798,172	6			$ 256,063	$ 195,563	5			$ 14,674
Arizona	1,256,099	1,113,686	1,079,056	13			199,472	162,370	2		3						
Arkansas	3,574,060	2,223,401	2,217,181	41	6	2	1,162,256	1,160,946	14		5	160,159	159,889	1		25	36,045
California	7,486,362	6,627,729	6,394,391	38	8		1,082,369	1,079,589	9			10,000	10,000	5			2,382
Colorado	2,631,567	1,482,761	1,429,513	19	1		881,478	850,333	9			377,010	339,301	3		21	12,420
Connecticut	1,712,684	297,379	297,379	2	1	1	792,481	771,120	5			616,150	596,965	3			47,220
Delaware	413,239	130,000	130,000	1			277,993	277,993	2								10,246
Florida	2,827,883	2,081,687	2,078,968	25	5	2	442,767	441,818	6			71,470	71,470			27	235,626
Georgia	4,895,949	148,437	146,945	4	3	1	1,251,771	1,251,771	26	6	4	1,026,000	1,026,000	16	3	47	2,471,233
Idaho	1,674,479	1,261,824	1,254,540	19	2	9	396,821	396,731	4		8	4,921	4,921			4	18,287
Illinois	10,307,184	7,123,011	7,096,719	57	6		2,998,945	2,998,945	16	2		161,000	161,000		2		50,520
Indiana	5,111,096	3,494,867	3,342,522	31	12		1,730,071	1,730,071	11		163						38,503
Iowa	5,600,679	3,651,048	3,561,663	82	9	7	1,967,638	1,964,911	25			80,407	68,690			1	5,415
Kansas	5,246,258	3,307,564	3,302,561	49		5	1,964,175	1,891,929	9	1		111,090	15,000	2			36,768
Kentucky	3,672,387	1,084,522	1,073,847	14	3		2,234,921	1,945,189	10	7		614,656	601,064	4	2		52,287
Louisiana	3,213,467	1,148,910	1,148,910	12	1		1,207,679	1,207,661	12	1		862,485	660,466	7	1		196,429
Maine	1,426,861	1,010,930	1,008,683	18		2	366,278	366,278	1	2		71,740	24,068	1		2	27,833
Maryland	2,061,751	418,680	418,680	3	3	16	804,729	804,729	4	1	3	617,561	545,093	5	1	7	293,249
Massachusetts	4,210,833	1,533,336	1,533,251	13	3		2,211,531	2,211,531	13	1		249,991	249,991	1			216,060
Michigan	6,765,197	5,981,247	5,787,280	43	6		969,077	898,608	1	2		43,500	43,500		1		35,809
Minnesota	5,395,441	4,146,084	4,027,069	75	11	39	1,365,677	1,354,524	10	2	11						13,849
Mississippi	3,241,475	1,592,932	1,592,768	39	4	2	1,123,568	1,123,568	16	3	5	40,100	40,100			14	485,039
Missouri	6,142,153	1,374,333	1,358,845	19			4,942,291	4,761,400	30	1	1	1,650	1,650			1	20,258
Montana	2,722,327	2,657,848	2,536,243	37	7		245,576	185,931	1	1							153
Nebraska	3,556,441	2,317,725	2,285,558	70	3	17	1,062,479	1,062,479	12		2	219,670	194,632	5		5	13,772
Nevada	887,260	885,820	859,960	8	3		13,308	13,308				3,630	3,630			5	10,362
New Hampshire	822,484	476,747	476,747	5	4		317,463	317,396	4		2	52,468	28,341	1		5	
New Jersey	3,983,826	1,017,054	1,017,054	9	2	1	2,646,639	2,635,594	13	7		291,410	291,410	2	1		39,768
New Mexico	1,725,286	1,678,000	1,672,314	18	1		25,879	25,879				56,505	11,202	1			15,891
New York	13,577,189	8,315,865	8,069,841	25	32		5,264,660	5,236,160	20	17		94,000	94,000				177,188
North Carolina	4,823,958	2,885,299	2,884,724	39	17		1,311,593	1,292,042	18	1	108	484,680	484,680	4			162,512
North Dakota	3,207,473	1,970,338	1,965,783	38	2		1,241,606	1,241,606	13	3							84
Ohio	8,439,897	1,441,897	1,364,395	9	3	2	5,852,726	5,494,403	36	5	1	1,284,879	1,184,174	12			396,925
Oklahoma	5,004,711	3,180,739	3,173,220	51	7	3	1,597,721	1,502,721	13	3	3	348,820	310,720	1	2	35	18,050
Oregon	2,334,204	2,239,057	2,154,389	15	6	2	177,027	174,122	2								5,693
Pennsylvania	11,483,613	3,872,046	3,512,325	46	14	9	8,053,908	7,534,712	36	9		150,000	150,000	4			286,575
Rhode Island	699,691	653,760	652,694	4	2		44,314	44,314		1							2,683
South Carolina	3,059,956	1,309,793	1,290,802	26	8	4	1,183,239	1,154,008	18	6	22	213,307	213,307	1	1	22	401,839
South Dakota	3,249,086	1,729,572	1,728,885	36	5	39	1,287,909	1,287,909	27	1	14	281,170	222,212	19			10,080
Tennessee	3,903,979	865,784	856,101	16	3	22	2,533,590	2,533,590	29		3	253,550	253,550	1	1	14	260,738
Texas	10,855,982	8,969,804	8,958,446	117	13	60	1,224,393	1,224,390	9	2	5	382,758	370,914		1	84	302,232
Utah	1,230,763	657,159	655,754	9	1	1	563,864	563,864	8								11,145
Vermont	729,857	561,433	534,633	7	6	17	1,204,133	187,274	3	1	3						7,950
Virginia	3,774,287	2,349,255	2,239,385	39	15	13	1,096,564	1,089,939	9	4		417,340	415,326	2	2	11	29,636
Washington	3,095,041	2,469,207	2,434,480	21	11	8	645,948	645,524	2	1	1	4,562	4,562			2	10,476
West Virginia	2,677,937	266,614	266,614	3		4	2,308,846	2,306,878	20	4	1	27,947	27,947	3		2	76,498
Wisconsin	5,022,683	3,601,206	3,565,228	33	5		1,426,581	1,426,537	4	2		68,667	29,367			19	1,550
Wyoming	1,360,841	886,875	886,767	10			467,369	467,368	4								6,706
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	293,667	292,776	3			226,162	158,370	2								2,556
TOTALS	196,000,000	112,245,858	110,067,576	1356	255	300	73,195,657	70,256,505	544	97	368	10,011,316	9,104,705	109	18	353	6,571,214

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Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

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Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

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Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

DEPARTMENT BULLETINS

No. 583D..Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 1279D..Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D..Highway Bridge Location. 15 cents.

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The Taxation of Motor Vehicles in 1932. 35 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y..Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF AUGUST 31, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 15,768,209	$ 8,305,736	$ 3,882,233	772.7	$ 72,922	$ 55,922	$ 17,000	3.9		$ 288,105	1.5	$ 8,475	$ 72,504
Arizona	5,211,960	2,641,935	9,036,003	5,205,763	2,616,771	537.0	19,573		19,573					6,197	5,591
Arkansas	6,748,335	3,423,049	11,094,422	6,714,187	3,409,259	626.4	47,750	33,944	13,586	10.9		3,200		204	2,005
California	15,607,354	7,932,206	30,592,038	15,607,354	7,738,760	763.9	114,733		109,877						83,569
Colorado	6,874,530	3,436,006	11,269,838	6,874,530	3,465,039	633.2	11,000		11,000						9,967
Connecticut	2,865,740	1,454,868	4,595,978	2,817,886	1,347,956	74.1	153,800	47,854	95,119	1.5					11,793
Delaware	1,819,088	923,395	2,782,832	1,818,804	916,149	128.9	8,210	284	7,246						
Florida	5,231,834	2,661,343	9,034,121	5,175,534	2,414,418	307.3	289,764	56,300	233,464	2.9					13,461
Georgia	10,091,185	5,113,491	13,873,270	9,412,325	3,589,643	794.7	905,006	237,969	667,037	41.0	$ 212,443	645,290	15.3	228,448	211,522
Idaho	4,486,249	2,277,486	7,125,725	4,486,249	2,199,313	501.5	74,779		49,236	1.9		11,428			17,508
Illinois	17,570,770	8,921,401	26,829,947	17,466,134	8,103,371	745.7	866,859	74,400	764,459	1.3		28,100	9.3	30,236	25,471
Indiana	10,037,843	5,088,963	15,749,737	10,017,034	4,861,544	485.1	52,340		52,340			58,728		20,809	116,352
Iowa	10,055,660	5,118,361	15,790,338	10,055,660	5,118,361	1,227.3									
Kansas	10,089,604	5,117,675	15,469,336	10,089,029	5,052,747	1,144.2	72,137		43,698	1.3				574	21,231
Kentucky	7,517,359	3,818,311	12,206,450	7,480,082	3,744,977	814.7	39,626		39,626	.8	11,306	17,973		25,971	15,735
Louisiana	5,828,591	2,963,932	9,132,363	5,755,288	2,688,031	258.9	395,131		235,959	18.4				73,303	39,942
Maine	3,369,917	1,711,586	5,329,092	3,363,872	1,702,680	195.0						8,644		6,045	262
Maryland	3,564,527	1,810,058	5,730,480	3,471,009	1,113,025	153.7	264,571	28,053	232,168	1.8	35,923	277,700	6.0	29,542	187,165
Massachusetts	6,597,100	3,350,474	10,510,393	6,550,778	3,139,337	115.5						91,944	.1	46,322	119,193
Michigan	12,736,227	6,452,568	20,772,274	12,736,227	6,295,890	768.1	179,788		136,788	.4					19,890
Minnesota	10,656,569	5,425,551	16,344,292	10,567,633	4,917,972	1,647.4	439,238		410,723	2.6	50,020			38,916	96,856
Mississippi	6,978,675	3,540,227	13,091,450	6,785,737	3,295,243	726.2	343,052	125,000	206,158	19.3	36,115	9,000	1.2	31,822	29,826
Missouri	12,180,306	6,173,740	18,317,246	12,119,584	5,136,421	1,446.4	984,283		984,283	1.1				60,722	53,036
Montana	7,439,748	3,769,734	11,764,611	7,425,285	3,655,874	1,058.4	131,534	6,905	57,239	.2				7,558	56,621
Nebraska	7,828,961	3,964,364	13,080,739	7,812,918	3,856,568	1,044.8	139,099	16,043	88,400	10.7					19,396
Nevada	4,545,917	2,302,356	7,073,517	4,545,917	2,266,964	758.8	21,979		21,979						13,413
New Hampshire	1,909,839	969,462	3,007,927	1,904,951	955,186	78.3								4,889	14,276
New Jersey	6,346,039	3,220,879	9,223,863	6,165,750	2,455,730	88.4	1,197,850	99,977	663,430	8.6	79,589	23,874	.5	723	77,844
New Mexico	5,792,935	2,941,700	8,912,013	5,748,150	2,922,863	750.0	55,585	29,694	10,800	1.2		2,695		15,091	5,342
New York	22,330,101	11,327,921	40,581,056	22,228,884	11,014,795	825.1	192,932	6,831	59,900	.1	6,600	124,900		87,786	128,326
North Carolina	9,522,293	4,840,941	15,320,720	9,364,174	4,721,698	1,357.6	238,501	143,020	93,171	40.9	5,960	8,200		9,139	17,872
North Dakota	5,804,448	2,938,967	8,927,781	5,736,109	2,406,584	2,151.7	112,386		112,386	14.9		199,691	3.9	68,339	220,305
Ohio	15,484,592	7,865,012	24,843,973	15,426,422	7,548,533	795.4	362,122	50,000	245,975	4.9				8,170	70,504
Oklahoma	9,216,798	4,685,180	14,782,680	9,214,165	4,449,353	806.6	232,864		232,663	.4				2,633	3,164
Oregon	6,106,896	3,097,814	10,077,336	6,102,835	2,963,472	469.5	46,467		46,467					4,062	87,875
Pennsylvania	18,891,004	9,590,788	29,418,947	18,589,519	9,130,548	1,057.9	743,047	256,582	421,025	13.5	12,000		1.0	32,903	39,215
Rhode Island	1,998,708	1,014,572	3,150,270	1,998,708	1,012,094	89.1	2,478		2,478						
South Carolina	5,459,165	2,770,954	8,221,105	5,368,486	2,519,597	625.8	277,234	217	242,149	10.6	40,000		.5	50,462	9,208
South Dakota	6,011,479	3,047,643	9,487,391	5,964,617	2,943,477	1,614.1	101,151	21,920	72,631	8.2	4,870	260	6.2	20,072	31,275
Tennessee	8,492,619	4,302,991	13,915,252	8,492,551	4,302,972	504.3								68	19
Texas	24,244,024	12,291,253	38,055,068	24,200,959	11,938,719	2,782.4	338,328	2,797	335,531	.4		2,050		40,268	14,953
Utah	4,194,708	2,132,691	7,459,642	4,194,708	2,132,691	590.9									
Vermont	1,867,573	948,007	3,180,278	1,867,573	948,007	141.0									
Virginia	7,416,757	3,765,387	11,591,319	7,326,822	3,381,572	629.1	232,166	26,448	200,086	20.7	38,186	171,333	22.8	25,301	12,396
Washington	6,115,867	3,106,412	9,407,780	6,112,042	3,020,316	303.0	46,596		46,596					3,825	39,500
West Virginia	4,474,234	2,280,335	6,457,136	4,308,274	1,804,365	212.9	378,997	54,892	301,059	9.1		168,139	1.7	111,068	6,772
Wisconsin	9,724,881	4,941,837	15,435,518	9,724,881	4,860,251	619.6	40,860		40,860			16,475			24,251
Wyoming	4,501,327	2,287,712	6,907,387	4,476,017	2,212,298	1,040.2	57,140	24,635	32,505	12.0		23,700	.9	675	19,208
District of Columbia	1,918,469	973,842	2,892,447	1,918,469	973,842	22.3									
Hawaii	1,871,062	949,778	3,142,645	1,857,812	725,649	54.8	175,648		172,480	1.1	13,250	26,799			24,850
TOTALS	394,000,000	200,000,000	636,764,235	390,953,433	187,873,158	35,339.9	10,459,526	1,399,687	7,829,150	266.6	546,262	2,208,228	70.9	1,100,618	2,089,464

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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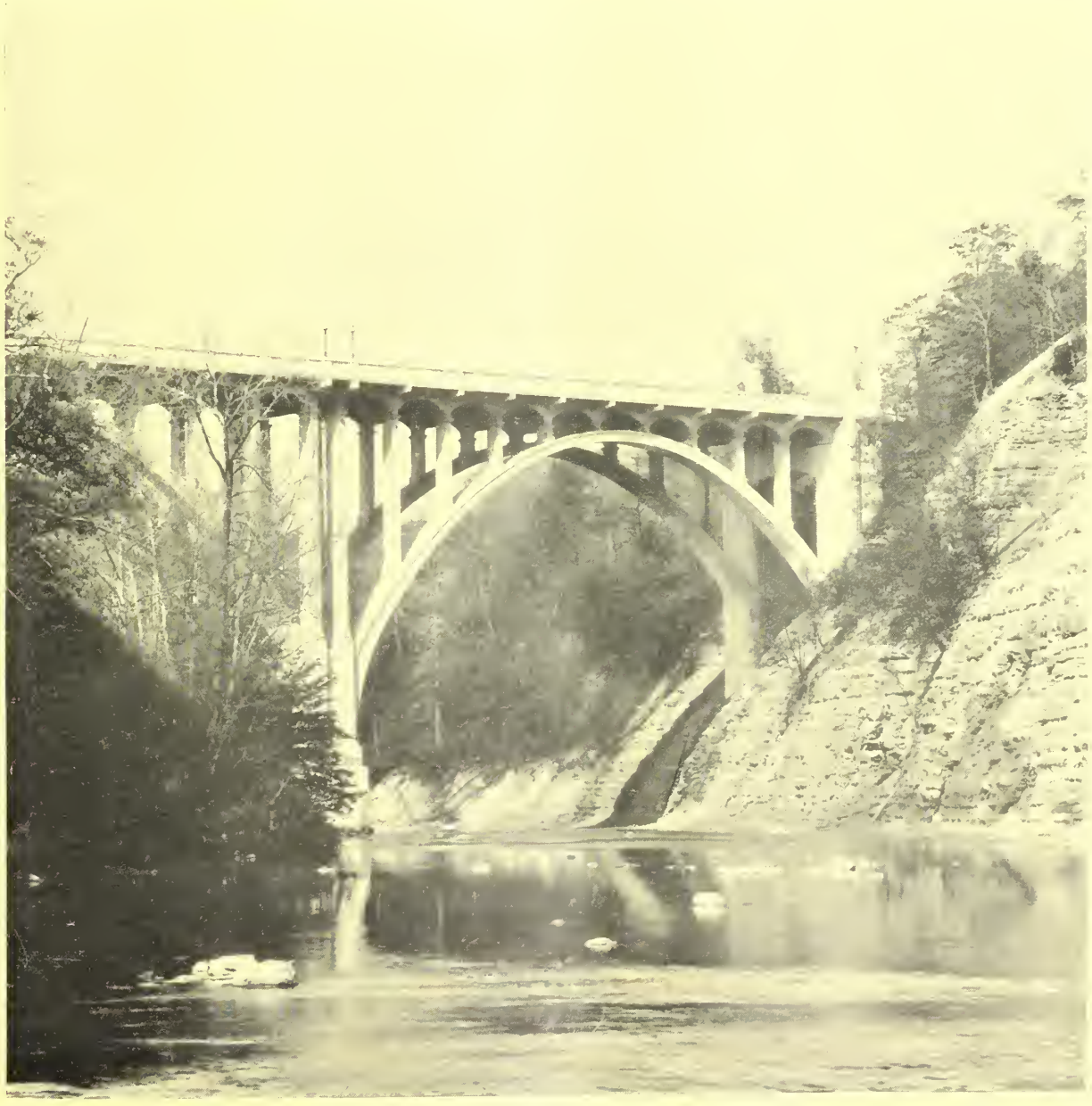

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 8 October 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

DISTRIBUTION OF WHEEL LOADS AND DESIGN OF REINFORCED CONCRETE BRIDGE FLOOR SLABS

Reported by H. R. ERPS and A. L. GOOGINS, Associate Highway Bridge Engineers, and J. L. PARKER, Highway Bridge Engineer, Bureau of Public Roads

SLABS for highway bridges are designed to support wheel loads in addition to the distributed dead loads. The methods now in use for estimating the influence of distributed dead loads are considered sufficiently accurate and will not be discussed in this study. The problem of stresses in slabs contributed by wheel loads is discussed in a paper, "Computation of Stresses in Bridge Slabs Due to Wheel Loads," by H. M. Westergaard, published in the March 1930 issue of PUBLIC ROADS.

In order to simplify design computations and provide for certain conditions not covered by the Westergaard formulas, there are advanced in this analysis formulas that are a modification of the Westergaard formulas. These formulas give results nearly the same as those proposed by Dr. Westergaard and will be referred to as the modified formulas.

Many of the formulas included in this paper were first developed by the late E. J. Budge, Associate Highway Bridge Engineer, Bureau of Public Roads.

Before any bridge floor slab can be designed, the type of live load and its manner of application must first be known or determined. The truck loading shown in figure 1 represents the loading considered in the subsequent analysis and design of reinforced concrete bridge floor slabs. This loading is the same as that shown in the Standard Specifications for Highway Bridges of the American Association of State Highway Officials, 1935, except that each wheel load is assumed to be applied to the slab over a circular area.

Two types of slabs will be considered, differing in the manner in which they are reinforced as follows:

Case I.—Slabs with main reinforcement parallel to the direction of traffic.

Case II.—Slabs with main reinforcement transverse to the direction of traffic.

The Westergaard investigation did not consider the effect of reinforcement, but was based on homogeneous and isotropic slabs, and the above and subsequent references to reinforcement under cases I and II are merely for the purpose of indicating the direction of the slab spans and the position of wheel loads on these spans.

When considering slabs with reinforcement parallel to the direction of traffic, the position of truck loading, for maximum moment, would be with the rear wheels on the center line of the slab for all spans less than 26.5 feet. This study is limited to spans from 2 to 25 feet, so the critical position of load will be with rear wheels on the center line of the slab.

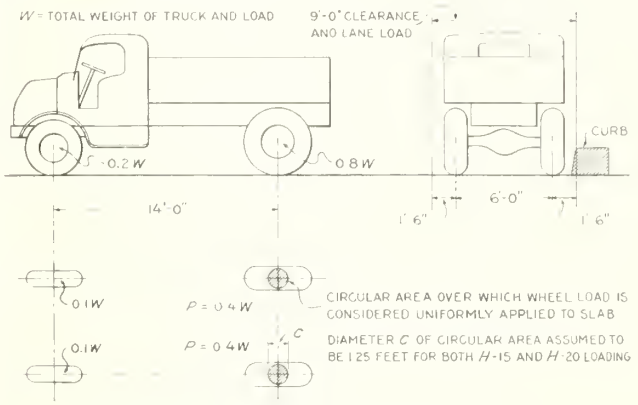

W = TOTAL WEIGHT OF TRUCK AND LOAD
9'-0" CLEARANCE AND LANE LOAD
0.2 W
0.8 W
CURB
14'-0"
6'-0"
1' 6"
1' 6"
0.1 W
0.1 W
P = 0.4 W
P = 0.4 W
C
CIRCULAR AREA OVER WHICH WHEEL LOAD IS CONSIDERED UNIFORMLY APPLIED TO SLAB
DIAMETER C OF CIRCULAR AREA ASSUMED TO BE 1.25 FEET FOR BOTH H-15 AND H-20 LOADING

FIGURE 1.—DIAGRAM OF TRUCK LOADING.

When considering slabs with reinforcement transverse to the direction of traffic, the study will be limited to slabs with spans from 2 to 10 feet. With slab spans of such lengths, the required roadway width is obtained by placing several slab spans together, so that there will be interior and exterior spans. On exterior spans, the maximum moment per foot width of slab will be produced by one rear wheel on the center line of the slab, because two wheels at 6-foot centers cannot be placed in position to give a greater moment on spans of less than 10.25 feet. On interior spans, the maximum moment per foot width of slab for the assumed loading will be produced by one rear wheel on the center line of the slab for spans less than 5.14 feet and by two wheels at 3-foot centers placed in position for maximum moment on spans from 5.14 to 10.25 feet.

Part I.—WESTERGAARD FORMULAS

The following Westergaard formulas for determining the live load bending moments in slabs were used to develop the modified formulas advanced in this analysis.

Slabs freely supported.—For slabs freely supported, the moment for one wheel on the center line of the slab (the center line of the slab is a line at the center of the slab span and parallel to the supports) is as follows:

$$M_{0x}=\frac{PS}{2.32\,S+8\,C}\quad\text{Equation (66)}\ ^{1}$$

M_{0x} = maximum moment, in foot-pounds per foot of width on the center line of a freely supported

[1] Equation numbers refer to those given in the article "Computation of Stresses in Bridge Slabs Due to Wheel Loads", by H. M. Westergaard, PUBLIC ROADS, vol. 11, no. 1, March 1930. The symbols s and c, given in lower case in the Westergaard article, have been changed to S and C, respectively, in this report.

slab, produced by a single load P on the center line of slab.

P=wheel load in pounds.

S=effective design span of slab in feet.

C=diameter of circle over which load P is considered uniformly applied to slab (to be taken as 1.25 feet for this study).

Moment for two or more wheels on the center line of freely supported slabs is considered next. Equation (66) gives the moment M_{0x} produced by one load P on the center line of the slab. If a second load, P_1, is placed on the center line of the slab at a distance y in feet from load P, the maximum moment per foot width of slab at load P will be $M_{0x}+M_x$.

$$M_x=0.21072\,P_1\log_{10}\coth\frac{\pi y}{2S}\pm\frac{0.2125\,P_1\,y}{S\sinh\frac{\pi y}{S}}\text{-Equation (74)}$$

M_x=the increase in moment M_{0x} caused by an additional load on the center line of slab at distance y from P.

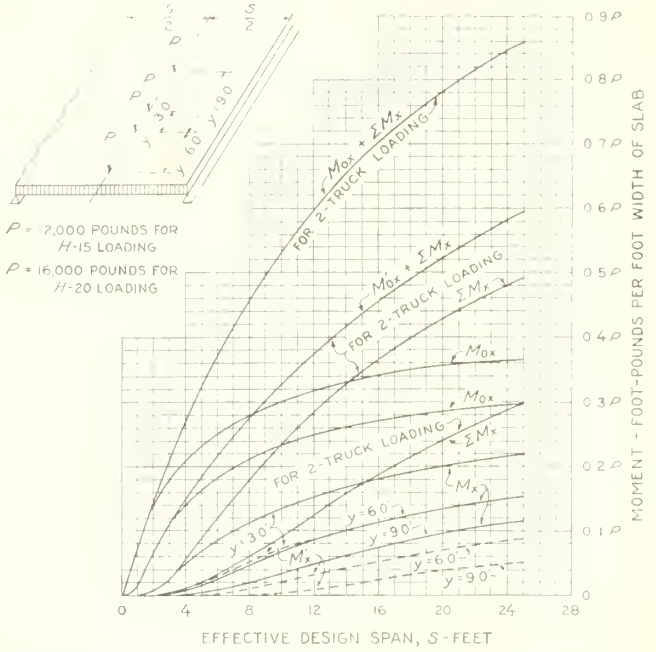


FIGURE 2.—BENDING MOMENTS PRODUCED BY TRUCK LOADINGS, FOR SLABS HAVING MAIN REINFORCEMENT PARALLEL TO THE DIRECTION OF TRAFFIC (CASE I). (VALUES OF M_{0x} AND M'_{0x} FOR CASE II ARE THE SAME AS FOR CASE I.)

When considering slabs with reinforcement parallel to the direction of traffic, the number of loads that can be placed on the center line of the slab depends on the width of roadway. When more than two loads are placed on the center line of the slab, the maximum moment per foot width of slab will be $M_{0x}+\Sigma M_x$. For slabs of less than 25-foot span, loads whose y distance from P is more than 9 feet produce very small values of M_x. Therefore, for computing the maximum moment per foot width of slab, two trucks will be considered sufficient for all widths of roadway.

The maximum moment for two wheels placed in the direction of the span on freely supported slabs is as follows:

$M_{0x}+\Delta M_x$=maximum moment per foot width of slab.

$$\frac{\Delta M_x}{P}=0.21072\log_{10}\frac{\cot\frac{\pi a}{4S}}{2}\text{ - - - - Equation (72)}$$

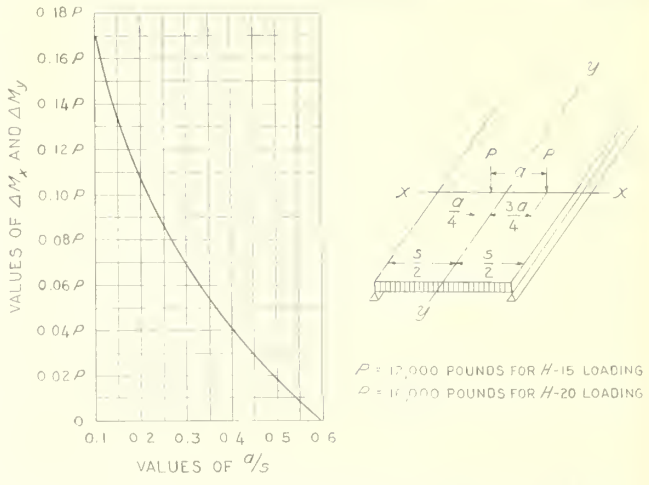


FIGURE 3.—CURVE SHOWING VALUES OF ΔM_x AND ΔM_y TO BE ADDED TO M_{0x} OR M_{0y} TO OBTAIN THE VALUES OF M_x OR M_y DUE TO THE COMBINED ACTION OF 2 LOADS P PLACED AS SHOWN, FOR SLABS HAVING MAIN REINFORCEMENT TRANSVERSE TO THE DIRECTION OF TRAFFIC.

ΔM_x=the increase in moment M_{0x} caused by placing two wheel loads on the span in position for maximum moment.

a=distance between loads.

For values of M_{0x} and of M_x, see figure 2. For values of ΣM_x and of $M_{0x}+\Sigma M_x$, when loading is limited to two trucks, see figure 2. For values of ΔM_x, see figure 3.

Slabs fully restrained.—For fully restrained slabs, the moment for one wheel on the center line of the slab is

$M'_{0x}=M_{0x}-0.0699\,P$ - - - - - - - - - - - - - - - - Equation (104)

M'_{0x}=maximum moment in foot-pounds per foot width, on the center line of a fully restrained slab, produced by a single load P on the center line of the slab.

Moment for two or more wheels on the center line of fully restrained slabs is considered next. Equation (104) gives the moment M'_{0x} produced by one load P on the center line of the slab. If a second load P_1 is placed on the center line of slab at a distance y in feet from P, the maximum moment per foot width of slab at load P will be $M'_{0x}+M'_x$.

$$M'_x-M_x=\frac{PS}{2l}\sum_{1,3,5,\ldots}^{n}\frac{\cos\omega_n y\,\tanh\alpha_n}{\sinh 2\alpha_n+2\alpha_n}[(1-\mu)\alpha_n\tanh\alpha_n-2]$$

- - - - - - - - - - - - - - - - -

the term ΔM_x is changed when the slab is fully restrained. It can be shown that the term ΔM_x is small compared to M_{0x} for short spans. If no reduction is applied to the term ΔM_x because of restraining the slab, the error will be small and on the safe side. So $M'_{0x}+\Delta M_x$ will be considered as the maximum moment per foot width of slab, for two wheels placed in the direction of the span in position for maximum moment when the slab is fully restrained.

For values of ΔM_x see figure 3.

Slabs partially restrained.—When slabs are continuous over supports, as shown in figures 4 and 5, they are neither freely supported nor totally fixed, but are partially restrained.

The Westergaard study does not advance formulas for partially restrained slabs; but since results for this condition fall between those for freely supported and totally fixed slabs, let slabs as in figure 4 and figure 5 be assumed to have 50 percent and 75 percent fixed end restraint, respectively. These assumptions are based upon the probable effect that continuity of the slab, yielding supports, and other influencing factors, may have on the degree of fixation at the supports. Then the moment for slabs as in figure 4 will be the average of

FIGURE 4.—CONTINUOUS SLAB ASSUMED TO HAVE 50 PERCENT FIXED END RESTRAINT.

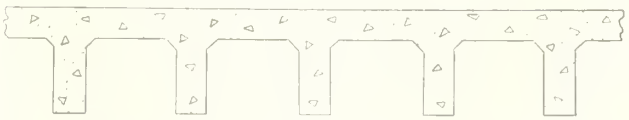


FIGURE 5.—CONTINUOUS SLAB ASSUMED TO HAVE 75 PERCENT FIXED END RESTRAINT.

moments for the simple span and for the totally fixed slab. The moment for slabs, as in figure 5, will be the moment for a simple span reduced by three-fourths the difference between the moments for a simple span and for a totally fixed slab.

Formulas for maximum moment for 50 and 75 percent fixed end restraint will be given in subsequent tables.

Part 2.—MODIFIED FORMULAS

The values obtained by the Westergaard formulas for 2-truck loading can be very closely approximated by the use of a factor E for the effective width over which the wheel load is distributed when only one wheel is involved, and by a factor H for the "equivalent effective width" when two or more wheels are involved. The width H is considered as the equivalent effective width for one wheel load to give the effect, in terms of maximum moment, of two or more wheels on the slab arranged for maximum bending.

The modified formulas have been developed for slabs with the two types of reinforcement, case I and case II.

Under cases I and II, modified formulas will be shown for slabs (1) freely supported, (2) fully restrained, (3) having 75 percent fixed end restraint, and (4) having 50 percent fixed end restraint.

Since the various modified formulas advanced in this study were developed by similar methods, only the method of development for slabs freely supported, case I, will be shown.

Slabs freely supported, case I.—The procedure in the development of the modified formulas for this case follows.

Figure 6 shows the moment values resulting from each of the four equal wheel loads, P_1, P_2, P_3, and P_4 at their respective points of application, by curves M_{0x}, and at points y distance from the wheels by curves M_x. The area under the curve ΣM_x, the total bending moment produced by the four wheels, is equal to PS. The maximum moment produced by the four wheels is $M_{0x}+\Sigma M_x$ and is under the wheel load having the greatest value of ΣM_x. The total moment area under curve ΣM_x can be represented by an equivalent rectangle with an ordinate $=M_{0x}+\Sigma M_x$ (the maximum moment produced on the slab) and an abscissa H_T (the effective width for the four wheels). The area of the rectangle is $(M_{0x}+\Sigma M_x)\times H_T=PS$ from which $H_T=\frac{PS}{(M_{0x}+\Sigma M_x)}$. Substituting the values of M_{0x} and ΣM_x obtained from Westergaard's equations (66) and (74), the value of H_T is found to be approximately $(0.66\ S+12.4)$ and $(M_{0x}+\Sigma M_x)=\frac{PS}{0.66\ S+12.4}$.

The total moment produced by one wheel is $\frac{PS}{4}$ and the area of the equivalent rectangle for one wheel is $(M_{0x}+\Sigma M_x)\times\frac{H_T}{4}=\frac{PS}{4}$. The equivalent effective width for one wheel that will give the effect, in terms of maximum moment of the four wheels, is then $H=\frac{H_T}{4}=\frac{0.66S+12.4}{4}=(0.165S+3.10)$ and $M_{0x}+\Sigma M_x=\frac{PS}{4H}=\frac{PS}{4(0.165S+3.10)}$. By substituting M for $M_{0x}+\Sigma M_x$ the modified formula for freely supported slabs with main reinforcement parallel to the direction of traffic becomes:

$$M=\frac{PS}{4H}=\frac{PS}{0.66\ S+12.4}\text{-----Modified formula}\quad(1)$$

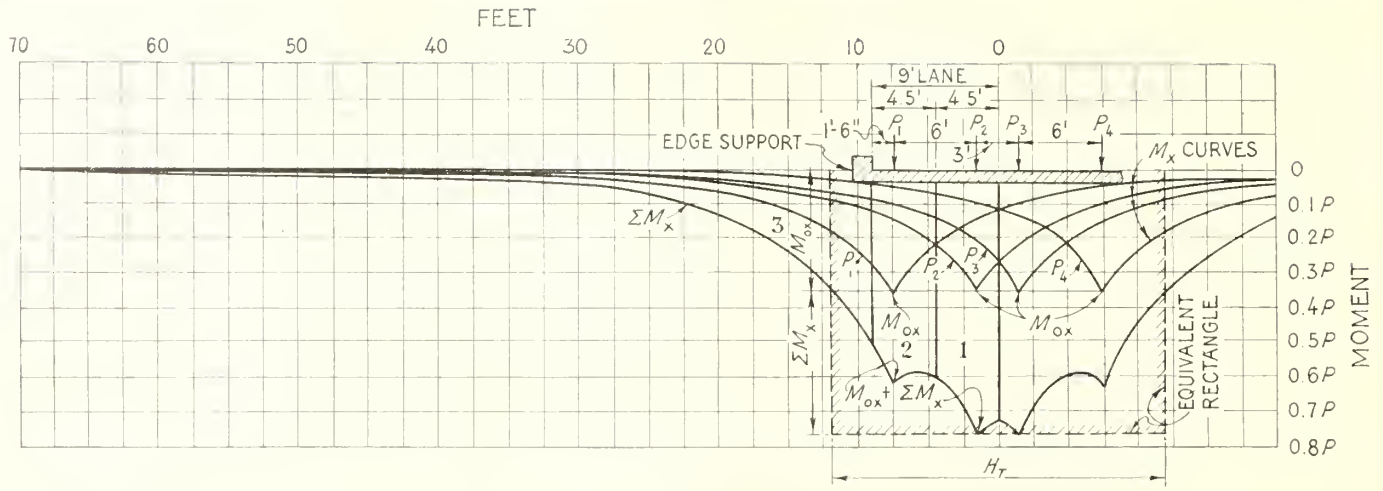
In like manner formulas for the remaining conditions of end support and end restraint under case I may be developed, the formulas being as follows:

Slabs fully restrained.—

$$M=\frac{PS}{8H}=\frac{PS}{0.66\ S+24.8}\text{-----Modified formula}\quad(2)$$

Slabs with 50 percent end restraint (as assumed for condition shown by fig. 4).—

$$M=\frac{PS}{5.5\ H}=\frac{PS}{0.66\ S+17.05}\text{--Modified formula}\quad(3)$$



The M_x curves show the variation in moment in a slab of unlimited width and 20-foot span for equal loads P_1, P_2, P_3, and P_4 and the summation of the M_x curves is shown by curve ΣM_x. When the slab has a limited width as shown by the superimposed sketch of slab, the inner 4.5 feet of the lane can be assumed to carry the moment represented by area 1; the outer 4.5 feet of lane is assumed to carry the moment represented by area 2, and the edge support is assumed to carry the moment represented by area 3

Moments represented by areas 1, 2, and 3 for 5-, 10-, 15-, 20-, and 25-foot spans

Span (feet)	Total moment distributed to various sections of slab			
	Inner 4.5 feet	Outer 4.5 feet	Edge support	9-foot lane plus edge support
5	1.2165 P	0.9765 P	0.3070 P	2.5 P
10	2.1500 P	1.7150 P	1.1350 P	5.0 P
15	2.8005 P	2.2995 P	2.4000 P	7.5 P
20	3.2760 P	2.7600 P	3.9640 P	10.0 P
25	3.6500 P	3.1275 P	5.7225 P	12.5 P

FIGURE 6.—DISTRIBUTION OF MOMENTS ALONG THE y-AXIS, SHOWING EQUIVALENT EFFECTIVE WIDTH OF DISTRIBUTION H_T; FOR A SLAB HAVING FREELY SUPPORTED ENDS, A 20-FOOT SPAN, 2 TRUCK LOADING, AND HAVING MAIN REINFORCEMENT PARALLEL TO THE DIRECTION OF TRAFFIC.

Slabs with 75 percent end restraint (as assumed for condition shown by fig. 5).—

$$M=\frac{PS}{6.5\ H}=\frac{PS}{0.66\ S+20.15}\text{—Modified formula} \quad (4)$$

Table 1 gives a summary of the modified and Westergaard formulas for maximum moment per foot width of slab, when reinforcement is parallel to direction of traffic (case I).

For a comparison of the moment values by the modified and Westergaard formulas, see figure 7.

TABLE 1.—*Westergaard and modified formulas for maximum moment*

Condition of slab at support	Westergaard formulas for moment values	Modified formulas giving equivalent moment values, M
Freely supported	$M_{0z}+\Sigma M_z$	$\frac{PS}{0.66\ S+12.4}$
50 percent end restraint	$\frac{M_{0z}+\Sigma M_z+M'_{0z}+\Sigma M'_z}{2}$	$\frac{PS}{0.66\ S+17.05}$
75 percent end restraint	$\frac{M_{0z}+\Sigma M_z+3M'_{0z}+3\Sigma M'_z}{4}$	$\frac{PS}{0.66\ S+20.15}$
Fully restrained	$M'_{0z}+\Sigma M'_z$	$\frac{PS}{0.66\ S+24.8}$

Edge support requirements.—For a wheel load placed on the center line of a slab, Westergaard's equation (74) shows the moment M_x produced on the center line of the slab at y distance from the load. Figure 8 shows that M_x is a maximum when the y distance is zero and that M_x is a minimum when the y distance is 3.0 S. Therefore, in slabs of unlimited width, as the span increases the width of slab in which moment is produced increases.

Figure 6 shows the moment M_x, at any point on the center line of a freely supported slab of unlimited width and 20-foot span, produced by each wheel for two trucks spaced 9 feet between centers. The curve ΣM_x shows the moment at any point in a slab of unlimited width produced by the four rear wheels, with a sketch of a slab of limited width superimposed. The total moment produced by the four wheels is $\frac{4\ PS}{4}$ or PS.

In a slab of limited width, when the loads are placed so that the outside load is 1 foot 6 inches from the face of the curb, a section of the slab and curb 9 feet wide is assumed to carry one half of the total moment produced by the four wheels, or $\frac{PS}{2}$. The moment $\frac{PS}{2}$ can be considered as distributed to the slab and edge support as shown in figure 6.

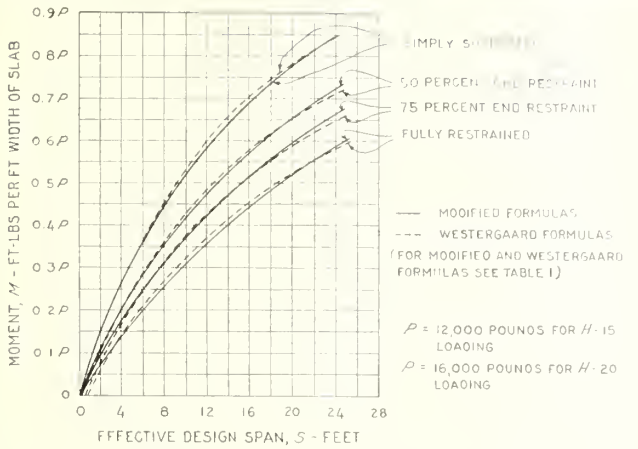

MOMENT, M - FT-LBS PER FT WIDTH OF SLAB
EFFECTIVE DESIGN SPAN, S - FEET
SIMPLY SUPPORTED
50 PERCENT END RESTRAINT
75 PERCENT END RESTRAINT
FULLY RESTRAINED
MODIFIED FORMULAS
WESTERGAARD FORMULAS
(FOR MODIFIED AND WESTERGAARD FORMULAS SEE TABLE 1)
P = 12,000 POUNDS FOR H-15 LOADING
P = 16,000 POUNDS FOR H-20 LOADING

FIGURE 7.—COMPARISON OF LIVE-LOAD BENDING MOMENTS PER FOOT WIDTH OF SLAB BY MODIFIED FORMULAS AND WESTERGAARD FORMULAS, FOR SLABS HAVING MAIN REINFORCEMENT PARALLEL TO THE DIRECTION OF TRAFFIC.

Let M_E be the moment, under the conditions indicated in figure 6, for which the edge support must provide. Then the modified formula for edge support requirement will be $M_E = 0.01\ PS^2$ for freely supported slabs.

The edge support requirement for slabs partially restrained and fully restrained may be provided for by using $0.8\ M_E$ or $0.008\ PS^2$ for 50 percent end restraint; $0.7\ M_E$ or $0.007\ PS^2$ for 75 percent end restraint; and $0.5\ M_E$ or $0.005\ PS^2$ for slabs fully restrained. The factors 1.0, 0.8, 0.7, and 0.5 represent approximately the variation in slab moments as computed by the modified formulas for the various conditions of support.

Reinforcement in direction of y-axis.—The previous discussion has dealt with moments in the direction of the span for which the main reinforcement is provided, referred to by Westergaard as moments in the direction of the x-axis. In addition to the moments in the direction of the slab span, there are moments produced in slabs normal to the span, referred to by Westergaard as moments in the direction of the y-axis, and for which reinforcement must be provided.

For a single load on the center line of a freely supported slab, the moment in the direction of the y-axis is given by Westergaard's equation (62) as

$$M_{0y} = M_{0x} - 0.0676\ P.$$

For two wheels on the center line of a freely supported slab, a distance y apart, the moment in the direction of the y-axis is $M_{0y} + M_y$. Values of M_y can be determined from Westergaard's equation (74).

For 2-truck loading with four rear wheels on the center line of the slab, when trucks are spaced 9 feet between centers, the moment in the direction of the y-axis is $M_{0y} + \Sigma M_y$, where ΣM_y is the summation of values of M_y for $y=3$ feet, $y=6$ feet, and $y=9$ feet. The values of M_y may be obtained from figure 8.

Figure 9 shows the maximum live-load moments plus impact moments in the direction of the y-axis, curves 1 and 2; the total moments for which main reinforcement must be provided are shown by curves 3 and 4; and the ratios of maximum moments in the direction of the y-axis to the total moments for which main reinforcement must be provided, expressed as percentages, are shown by curves 5 and 6.

The moments along the y-axis are small at points near the supports, and the maximum occurs at the center. Since the variations in percentages shown in figure 9 are not large, it is recommended that, for slabs with main reinforcement parallel to the direction of traffic, the bottom of the slab, in the direction transverse to traffic, be reinforced with the percentages of main reinforcement shown in table 2, where practicable.

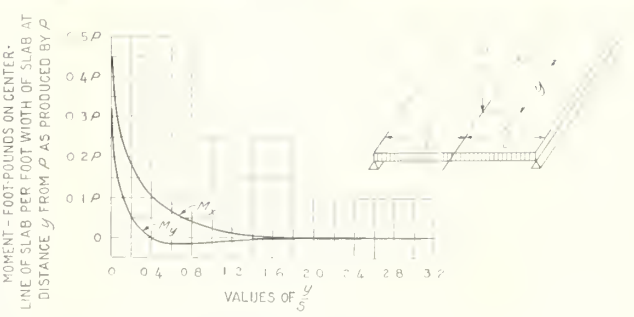


FIGURE 8.—BENDING MOMENTS ON CENTER LINE OF SIMPLY SUPPORTED SLAB AT DISTANCE y FROM P AS PRODUCED BY P, FOR SLABS HAVING MAIN REINFORCEMENT PARALLEL TO THE DIRECTION OF TRAFFIC.

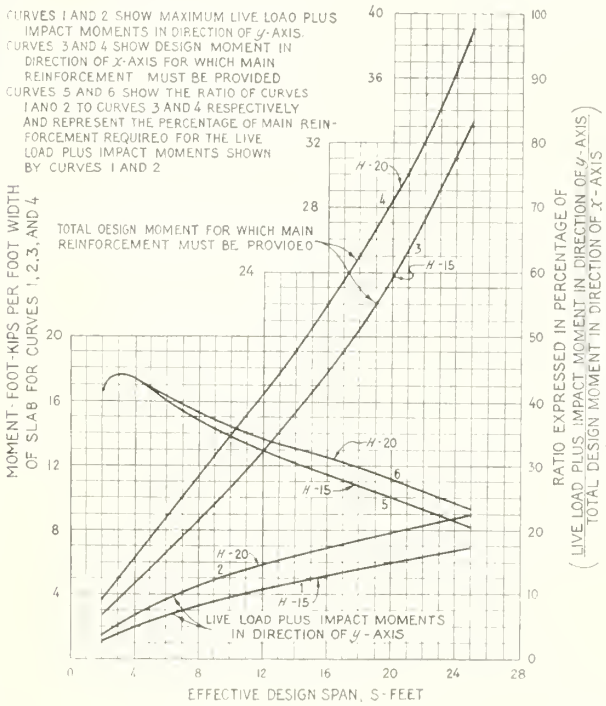

CURVES 1 AND 2 SHOW MAXIMUM LIVE LOAD PLUS IMPACT MOMENTS IN DIRECTION OF y-AXIS.
CURVES 3 AND 4 SHOW DESIGN MOMENT IN DIRECTION OF x-AXIS FOR WHICH MAIN REINFORCEMENT MUST BE PROVIDED
CURVES 5 AND 6 SHOW THE RATIO OF CURVES 1 AND 2 TO CURVES 3 AND 4 RESPECTIVELY AND REPRESENT THE PERCENTAGE OF MAIN REINFORCEMENT REQUIRED FOR THE LIVE LOAD PLUS IMPACT MOMENTS SHOWN BY CURVES 1 AND 2
TOTAL DESIGN MOMENT FOR WHICH MAIN REINFORCEMENT MUST BE PROVIDED
MOMENT - FOOT-KIPS PER FOOT WIDTH OF SLAB FOR CURVES 1, 2, 3, AND 4
RATIO EXPRESSED IN PERCENTAGE OF (LIVE LOAD PLUS IMPACT MOMENT IN DIRECTION OF y-AXIS / TOTAL DESIGN MOMENT IN DIRECTION OF x-AXIS) AS SHOWN BY CURVES 5 AND 6
H-20
H-15
LIVE LOAD PLUS IMPACT MOMENTS IN DIRECTION OF y-AXIS
EFFECTIVE DESIGN SPAN, S - FEET

FIGURE 9.—PERCENTAGE OF MAIN REINFORCEMENT REQUIRED IN BOTTOM OF SLAB IN DIRECTION OF y-AXIS, FOR SLABS HAVING MAIN REINFORCEMENT PARALLEL TO THE DIRECTION OF TRAFFIC.

TABLE 2.—*Percentages of main reinforcement recommended for the bottom of the slab, in the direction transverse to traffic*

Span length	In middle half of span	In outer quarters of span
	Percent	*Percent*
Under 10 feet	45	30
10 to 20 feet	35	25
Over 20 feet	25	15

Bridge floor slabs with main reinforcement transverse to the direction of traffic usually consist of several spans continuous on stringers or built monolithic with the supporting girders. For slab spans so arranged, the

maximum moment per foot width of slab in exterior spans will be produced by one rear wheel on the center line of the span for spans up to 10 feet. On interior spans, the maximum moment per foot width of slab will be produced by one rear wheel on the center line of the span for spans up to 5.14 feet, and by two rear wheels placed 3 feet apart for spans from 5.14 to 10 feet.

In order to simplify the moment computations and also to provide for the possibility of loads spaced less than 3 feet apart, the modified formulas for interior spans loaded with two wheels are developed for slab spans from 4 feet to 10 feet.

The modified formulas for computing the live-load bending moments per foot width of slab for slabs with main reinforcement transverse to the direction of traffic are developed in a manner similar to that used for slabs freely supported of case I, and are as follows:

Slabs freely supported, case II.—Spans less than 4 feet.

$$M=\frac{PS}{4E}=\frac{P\sqrt{S}}{9.64}\text{-----Modified formula (5)}$$

Exterior spans 4 to 10 feet,

$$M=\frac{PS}{4E}=\frac{PS}{2.32\,S+10.0}\text{----Modified formula (6)}$$

Interior spans 4 to 10 feet,

$$M=\frac{PS}{4I}=\frac{PS}{1.32\,S+14.0}\text{----Modified formula (7)}$$

Slabs fully restrained.—Spans less than 4 feet,

$$M=\frac{P\sqrt{S}}{9.64}-0.07\,P\text{-------Modified formula (8)}$$

Exterior spans 4 to 10 feet,

$$M=\frac{PS}{2.32\,S+10.0}-0.07\,P\text{-Modified formula (9)}$$

Interior spans 4 to 10 feet,

$$M=\frac{PS}{1.32\,S+14.0}-0.07\,P\text{-Modified formula (10)}$$

Slabs with 50 percent end restraint (as assumed for condition shown by figure 4).—Spans less than 4 feet,

$$M=\frac{P\sqrt{S}}{9.64}-0.035\,P\text{------Modified formula (11)}$$

Exterior spans 4 to 10 feet,

$$M=\frac{PS}{2.32\,S+10.0}-0.035\,P\text{-Modified formula (12)}$$

Interior spans 4 to 10 feet,

$$M=\frac{PS}{1.32\,S+14.0}-0.035\,P\text{-Modified formula (13)}$$

Slabs with 75 percent end restraint (as assumed for condition shown by figure 5).—Spans less than 4 feet,

$$M=\frac{P\sqrt{S}}{9.64}-0.0525\,P\text{-----Modified formula (14)}$$

Exterior spans 4 to 10 feet,

$$M=\frac{PS}{2.32\,S+10.0}-0.0525P\text{-Modified formula (15)}$$

Interior spans 4 to 10 feet,

$$M=\frac{PS}{1.32\,S+14.0}-0.0525\,P\text{-Modified formula (16)}$$

Table 3 gives a summary of the Westergaard formulas for interior and exterior spans. Table 4 gives a summary of the modified formulas for interior and exterior spans.

Figure 10 shows a comparison of the moment values by the modified and Westergaard formulas for interior spans, and figure 11 shows a comparison for exterior spans.

Requirements at unsupported edges.—For slabs with main reinforcement transverse to the direction of traffic, where the continuity of the slab is broken, the edges of the slab require support by diaphragms or other suitable means.

TABLE 3.—*Summary of Westergaard formulas for interior and exterior spans*

Condition of slab at support	Span	Exterior spans	Interior spans
	Feet		
Freely supported	2 to 10	M_{0x}	
	2 to 5.14		M_{0x}
	5.14 to 10		$M_{0x}+\Delta M_x$
Fully restrained	2 to 10	M'_{0x}	
	2 to 5.14		M'_{0x}
	5.14 to 10		$M'_{0x}+\Delta M_x$
50 percent end restraint	2 to 10	$\frac{M_{0x}+M'_{0x}}{2}$	
	2 to 5.14		$\frac{M_{0x}+M'_{0x}}{2}$
	5.14 to 10		$\frac{M_{0x}+M'_{0x}+2\Delta M_x}{2}$
75 percent end restraint	2 to 10	$\frac{M_{0x}+3M'_{0x}}{4}$	
	2 to 5.14		$\frac{M_{0x}+3M'_{0x}}{4}$
	5.14 to 10		$\frac{M_{0x}+3M'_{0x}+4\Delta M_x}{4}$

TABLE 4.—*Summary of modified formulas for interior and exterior spans*

Condition of slab at support	Span	Exterior spans	Interior spans
	Feet		
Freely supported	0 to 4	$M=\frac{P\sqrt{S}}{9.64}$	$M=\frac{P\sqrt{S}}{9.64}$
	4 to 10	$M=\frac{PS}{2.32S+10}$	$M=\frac{PS}{1.32S+14}$
Fully restrained	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.07P$	$M=\frac{P\sqrt{S}}{9.64}-0.07P$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.07P$	$M=\frac{PS}{1.32S+14}-0.07P$
50 percent end restraint	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.035P$	$M=\frac{P\sqrt{S}}{9.64}-0.035P$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.035P$	$M=\frac{PS}{1.32S+14}-0.035P$
75 percent end restraint	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.0525P$	$M=\frac{P\sqrt{S}}{9.64}-0.0525P$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.0525P$	$M=\frac{PS}{1.32S+14}-0.0525P$

Reinforcement in direction of y-axis.—Since the slabs for case II are limited to spans of 10 feet and less, the maximum moment M_{0y} in the direction of the y-axis will be the moment caused by one wheel load on the center line of the slab for exterior spans.

$$M_{0y}=M_{0x}-0.0676\,P\text{---(Equation 62)}$$

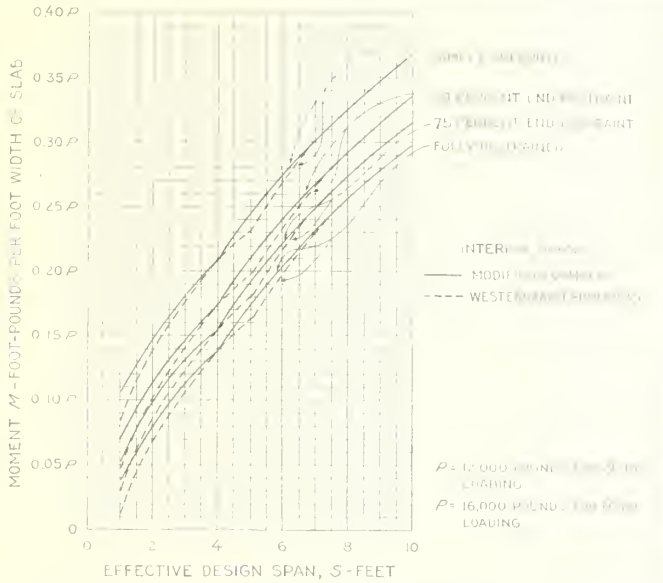


FIGURE 10.—COMPARISON OF LIVE-LOAD BENDING MOMENTS IN INTERIOR SPANS PER FOOT WIDTH OF SLAB BY MODIFIED AND WESTERGAARD FORMULAS, FOR SLABS HAVING MAIN REINFORCEMENT TRANSVERSE TO THE DIRECTION OF TRAFFIC.

The maximum moment M_{0y} in the direction of the y-axis for interior spans will be produced by one load on the center line of slab for slabs with spans less than 5.14 feet. For slabs with spans from 5.14 feet to 10 feet, the maximum moment in the direction of the y-axis will be produced by two wheels 3 feet apart placed in the direction of the span. For this position of loads, the moment in the direction of y will be $M_{0y}+\Delta M_y$. Values of ΔM_y may be obtained from figure 3.

Figure 12 shows the maximum live-load moments plus impact moments in the direction of the y-axis, curves 1 and 2; the total moments in the direction of the x-axis for which the main reinforcement must be provided are shown by curves 3 and 4; and the ratio of maximum moments in the direction of y to total moments in the direction of x is shown by curve 5. Since curve 5 shows the moment in the direction of y expressed in percentage of the moment in the direction of x, the same curves could be used to determine the percentage of main steel to be used in the direction of y.

The maximum percentage of main reinforcement required in the direction of the y-axis is shown by curve 5, figure 12, to vary from 50 to 69 percent. For practical application it is recommended that the following percentages of main reinforcement be provided per foot width of slab in the direction of the y-axis.

Element of slab:	*Percentage of main reinforcement per foot width of slab*
Middle half of slab span	65
Outer quarters of slab span	45

The reinforcement required in the direction of the y-axis is to be used in the bottom of the slab and is in addition to any temperature reinforcement used in the top of the slab.

Provision for negative moments at supports.—The modified formulas are for moments at or near the center line of the span. The bridge floor slabs, except those freely supported, are designed to have practically the same moment of resistance (negative) over the supports as (positive) at the middle of the span.

Cantilever slabs.—The modified formulas developed in this analysis apply only to slabs adequately supported at the sides and ends and do not apply to cantilevered portions of slabs or where cantilevered portions affect the slab.

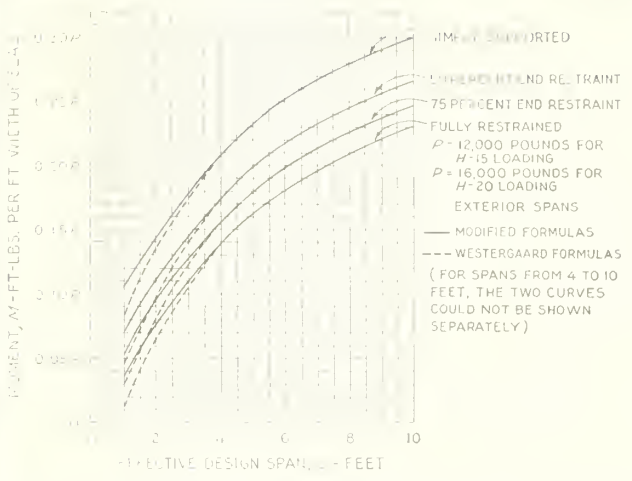


FIGURE 11.—COMPARISON OF LIVE-LOAD BENDING MOMENTS IN EXTERIOR SPANS PER FOOT WIDTH OF SLAB BY MODIFIED AND WESTERGAARD FORMULAS, FOR SLABS HAVING MAIN REINFORCEMENT TRANSVERSE TO THE DIRECTION OF TRAFFIC.

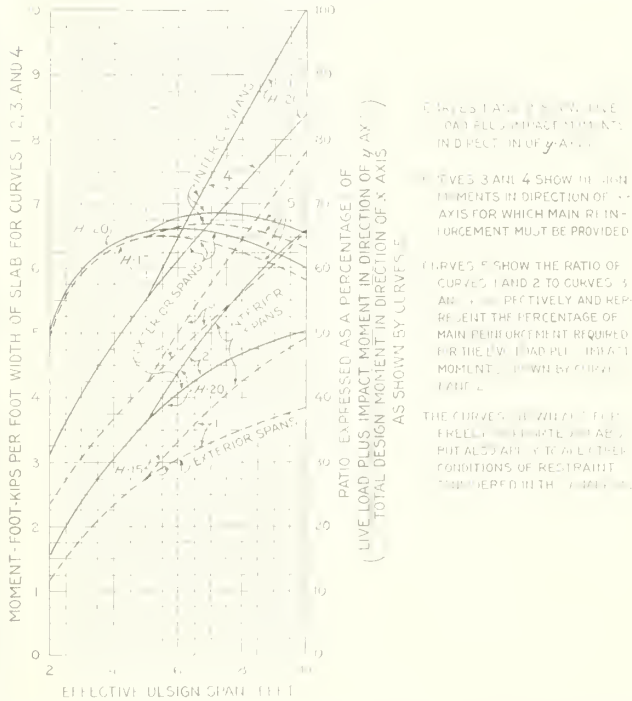

MOMENT-FOOT-KIPS PER FOOT WIDTH OF SLAB FOR CURVES 1, 2, 3, AND 4
PERCENTAGE OF (LIVE LOAD PLUS IMPACT MOMENT IN DIRECTION OF y AXIS / TOTAL DESIGN MOMENT IN DIRECTION OF x AXIS) AS SHOWN BY CURVE 5
INTERIOR SPANS
EXTERIOR SPANS
H-20
H-15
EFFECTIVE DESIGN SPAN, FEET

FIGURE 12.—PERCENTAGE OF MAIN REINFORCEMENT REQUIRED IN BOTTOM OF SLABS IN DIRECTION OF THE y-AXIS, FOR SLABS HAVING MAIN REINFORCEMENT TRANSVERSE TO THE DIRECTION OF TRAFFIC. (Also see tabulation on p. 155.)

Part 3.—DESIGN OF REINFORCED CONCRETE BRIDGE FLOOR SLABS BY MODIFIED FORMULAS

Case I.—Modified formulas will be applied in the design of reinforced concrete bridge floor slabs with main reinforcement parallel to the direction of traffic.

The 1935 specifications of the American Association of State Highway Officials will be used, modified as follows:

Live load moments are computed according to modified formulas as shown in table 5.

The outer edges of the slab shall be supported by a beam or extra slab width capable of resisting live-load edge support moment M_E, live-load impact and dead-load moment.

Unit stresses: f'_c=3,000 pounds per square inch; f_s=18,000 pounds per square inch; f_c=800 pounds per square inch; n=12.

Live load: *H*–15 and *H*–20 truck train loading, on two adjacent lanes for all roadway widths.

Symbols used are defined as follows:

S=Effective design span in feet.
W=Dead load per square foot of slab.
P=Wheel load=12,000 pounds for *H*–15 and 16,000 pounds for *H*–20 loading.

$$I=\text{Impact factor}=\frac{50}{125+S}$$

M=Maximum live load moment.
$M(1.0+I)$=Live load+impact moment.
M_t=Design moment=Dead load moment+$M(1.0+I)$.
M_E=Live-load moment in foot-pounds distributed to edge support.
$M_E(1.0+I)$=Live load+impact moment in foot-pounds distributed to edge support.

All moments except for M_E (edge support) are in foot-pounds per foot width of slab.

For values of live-load moments in foot-pounds per foot width of slab by above formulas, see tables 6 and 8.

For values of live-load moments distributed to edge support, see tables 7 and 9.

Tables 10 to 17 show design moments, dimensions nd reinforcement of slabs for case I.

TABLE 6.—*Live load moments in foot-pounds per foot width of slab*

Controlling conditions:
H–15 loading.
P=12,000 pounds.
Case I.—parallel reinforcement.
Modified formulas.
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

Condition at support		Freely supported		Slabs continuous, 50 percent end restraint		Slabs monolithic, 75 percent end restraint		Fully restrained	
Slab span S, feet	$I=\frac{50}{125+S}$	$M=\frac{PS}{0.66S+12.4}$		$M=\frac{PS}{0.66S+17.05}$		$M=\frac{PS}{0.66S+20.15}$		$M=\frac{PS}{0.66S+24.8}$	
S	I	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$
2.0	0.394	1,750	2,440	1,310	1,830	1,120	1,560	920	1,280
2.5	.392	2,140	2,980	1,600	2,230	1,380	1,920	1,130	1,570
3.0	.391	2,510	3,480	1,890	2,630	1,630	2,270	1,340	1,865
3.5	.390	2,850	3,960	2,170	3,020	1,870	2,600	1,550	2,150
4.0	.388	3,190	4,430	2,440	3,390	2,100	2,920	1,750	2,430
4.5	.386	3,520	4,880	2,700	3,740	2,330	3,230	1,950	2,700
5.0	.385	3,830	5,300	2,950	4,090	2,560	3,540	2,140	2,965
5.5	.383	4,110	5,680	3,190	4,410	2,770	3,830	2,330	3,220
6.0	.382	4,400	6,080	3,430	4,740	2,990	4,130	2,510	3,470
6.5	.380	4,680	6,460	3,650	5,040	3,190	4,400	2,680	3,700
7.0	.379	4,940	6,810	3,870	5,340	3,390	4,670	2,850	3,930
7.5	.377	5,180	7,140	4,090	5,630	3,590	4,950	3,030	4,170
8.0	.376	5,430	7,460	4,300	5,920	3,780	5,200	3,200	4,400
8.5	.375	5,670	7,800	4,500	6,190	3,960	5,450	3,360	4,620
9.0	.373	5,890	8,085	4,700	6,450	4,130	5,670	3,520	4,830
9.5	.372	6,100	8,370	4,890	6,710	4,310	5,920	3,670	5,040
10.0	.370	6,310	8,645	5,070	6,950	4,480	6,140	3,820	5,240
11.0	.368	6,700	9,165	5,420	7,420	4,810	6,580	4,130	5,650
12.0	.365	7,090	9,680	5,770	7,870	5,130	7,000	4,400	6,010
13.0	.362	7,440	10,140	6,080	8,290	5,410	7,370	4,660	6,350
14.0	.360	7,760	10,550	6,390	8,690	5,700	7,750	4,940	6,720
15.0	.357	8,080	10,960	6,670	9,060	5,980	8,100	5,180	7,030
16.0	.355	8,360	11,330	6,950	9,420	6,240	8,460	5,430	7,360
17.0	.352	8,630	11,670	7,210	9,760	6,500	8,780	5,670	7,660
18.0	.350	8,890	12,000	7,460	10,080	6,720	9,070	5,880	7,940
19.0	.347	9,150	12,330	7,700	10,380	6,950	9,370	6,110	8,230
20.0	.345	9,370	12,600	7,930	10,670	7,180	9,640	6,320	8,500
21.0	.342	9,600	12,880	8,140	10,940	7,390	9,900	6,530	8,760
22.0	.340	9,810	13,150	8,350	11,190	7,600	10,180	6,720	9,000
23.0	.338	9,990	13,370	8,560	11,460	7,790	10,430	6,900	9,240
24.0	.336	10,200	13,630	8,740	11,700	7,980	10,660	7,060	9,440
25.0	.333	10,380	13,840	8,930	11,920	8,160	10,880	7,270	9,700

TABLE 5. *Summary of modified formulas for case* I

Type of slab	Requirements regarding slab and supports	Percentage of end restraint assumed	Live load moments per foot width of slab, M	Live load moment distributed to edge support, M_E	Dead load moment
Freely supported	Single spans	0	$\frac{PS}{0.66S+12.4}$	$0.01\ PS^2$	$\frac{WS^2}{8}$
Slabs continuous	Reinforced concrete slabs bearing on 3 or more supports and continuous over 2 or more panels	50	$\frac{PS}{0.66S+17.05}$	$0.008\ PS^2$	$\frac{WS^2}{10}$
Slabs monolithic	Reinforced concrete slabs built monolithic with 3 or more supports and continuous over 2 or more panels	75	$\frac{PS}{0.66S+20.15}$	$0.007\ PS^2$	$\frac{WS^2}{10}$
Fully restrained	Exceptional cases only	100	$\frac{PS}{0.66S+24.8}$	$0.005\ PS^2$	$\frac{WS^2}{12}$

TABLE 7.—*Live load moments in foot-pounds distributed to edge supports*

Controlling conditions:
H-15 loading.
P=12,000 pounds
Case I.—parallel reinforcement.
Modified formulas.
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

Condition at support		Freely supported		Slabs continuous, 50 percent end restraiut		Slabs monolithic, 75 percent end restraint		Fully restrained	
Slab span S, feet	$I=\frac{50}{125+S}$	$M_E=0.01\ PS^2$		$M_E=0.008\ PS^2$		$M_E=0.007\ PS^2$		$M_E=0.005\ PS^2$	
S	I	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$
2.0	0.394	480	670	385	535	335	470	240	335
2.5	.392	750	1,045	600	835	525	730	375	525
3.0	.391	1,080	1,500	865	1,200	755	1,050	540	750
3.5	.390	1,470	2,040	1,175	1,640	1,030	1,425	735	1,020
4.0	.388	1,920	2,665	1,540	2,125	1,345	1,865	960	1,330
4.5	.386	2,430	3,370	1,945	2,700	1,700	2,365	1,215	1,690
5.0	.385	3,000	4,160	2,400	3,325	2,100	2,919	1,500	2,080
5.5	.383	3,630	5,030	2,900	4,025	2,540	3,515	1,815	2,510
6.0	.382	4,320	5,975	3,465	4,775	3,025	4,185	2,160	2,990
6.5	.380	5,070	7,000	4,065	5,600	3,550	4,900	2,535	3,500
7.0	.379	5,880	8,110	4,700	6,475	4,125	5,675	2,940	4,050
7.5	.377	6,750	9,300	5,400	7,440	4,725	6,510	3,375	4,650
8.0	.376	7,680	10,560	6,140	8,450	5,375	7,400	3,840	5,280
8.5	.375	8,670	11,920	6,940	9,550	6,075	8,350	4,335	5,960
9.0	.373	9,720	13,330	7,770	10,680	6,800	9,350	4,860	6,660
9.5	.372	10,830	14,850	8,670	11,880	7,570	10,400	5,415	7,430
10.0	.370	12,000	16,440	9,600	13,150	8,400	11,500	6,000	8,220
11.0	.368	14,520	19,880	11,620	15,890	10,170	13,920	7,260	9,940
12.0	.365	17,280	23,620	13,820	18,880	12,100	16,530	8,640	11,810
13.0	.362	20,280	27,630	16,250	22,100	14,180	19,350	10,140	13,820
14.0	.360	23,520	32,000	18,820	25,600	16,460	22,370	11,760	16,000
15.0	.357	27,000	36,640	21,620	29,280	18,880	25,620	13,500	18,320
16.0	.355	30,720	41,640	24,570	33,300	21,500	29,100	15.360	20,820
17.0	.352	34,689	46,890	27,750	37,500	24,320	32,850	17,340	23,450
18.0	.350	38,880	52,500	31,120	42,000	27,250	36,750	19,440	26,250
19.0	.347	43,320	58,400	34,660	46,700	30,369	40,850	21,660	29,200
20.0	.345	48,000	64,600	38,370	51,650	33,600	45,209	24,000	32,300
21.0	.342	52,920	71,000	42,380	56,760	37,060	49,750	26,463	35,500
22.0	.340	58,080	77,800	46,480	62,280	40,700	54,500	29,040	38,900
23.0	.338	63,480	85.000	50,759	68,000	44,450	59,500	31,740	42,500
24.0	.336	69,120	92,400	55.350	73,880	48,360	64,700	34,560	46,200
25.0	.333	75,000	100,000	60,000	89,000	52,500	70.000	37,500	50,000

TABLE 8.—*Live load moments in foot-pounds per foot width of slab*

Controlling conditions:
H-20 loading.
P = 16,000 pounds.
Case I—parallel reinforcement.
Modified formulas.
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

Condition at support		Freely supported		Slabs continuous, 50 percent end restraint		Slabs monolithic, 75 percent end restraint		Fully restrained	
Slab span S, feet	$I=\frac{50}{125+S}$	$M=\frac{PS}{0.66S+12.4}$		$M=\frac{PS}{0.66S+17.05}$		$M=\frac{PS}{0.66S+20.15}$		$M=\frac{PS}{0.66S+24.8}$	
S	I	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$
2.0	0.394	2,330	3,250	1,750	2,440	1,490	2,080	1,230	1,710
2.5	.392	2,850	3,970	2,130	2,970	1,840	2,560	1,510	2,100
3.0	.391	3,340	4,640	2,520	3,510	2,170	3,030	1,790	2,490
3.5	.390	3,800	5,280	2,890	4,030	2,490	3,470	2,070	2,870
4.0	.388	4,250	5,900	3,250	4,520	2,800	3,890	2,330	3,240
4.5	.386	4,690	6,500	3,600	4,980	3,110	4,310	2,590	3,600
5.0	.385	5,100	7,060	3,930	5,450	3,410	4,720	2,850	3,950
5.5	.383	5,490	7,580	4,250	5,880	3,700	5,110	3,100	4,290
6.0	.382	5,870	8,110	4,570	6,320	3,980	5,500	3,340	4,630
6.5	.380	6,240	8,620	4,870	6,720	4,250	5,860	3,580	4,930
7.0	.379	6,570	9,080	5,160	7,120	4,520	6,220	3,810	5,240

TABLE 8.—*Live load moments in foot-pounds per foot width of slab*—Continued

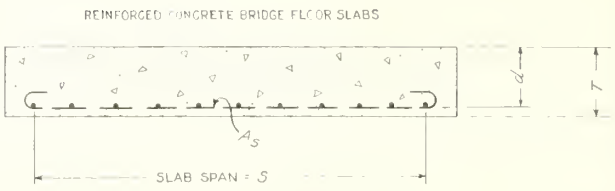
Condition at support		Freely supported		Slabs continuous, 50 percent end restraint		Slabs monolithic, 75 percent end restraint		Fully restrained	
Slab span S, feet	$I=\frac{50}{125+S}$	$M=\frac{PS}{0.66S+12.4}$		$M=\frac{PS}{0.66S+17.05}$		$M=\frac{PS}{0.66S+20.15}$		$M=\frac{PS}{0.66S+24.8}$	
S	I	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$
7.5	0.377	6,910	9,520	5,450	7,510	4,790	6,600	4,010	5.560
8.0	.376	7,240	9,940	5,730	7,900	5,040	6,930	4,260	5.860
8.5	.375	7,560	10,400	6,000	8,250	5,280	7,270	4,470	6.160
9.0	.373	7,840	10,770	6,270	8,600	5,510	7,560	4,690	6.440
9.5	.372	8,140	11,150	6,520	8,940	5,750	7,890	4,900	6,720
10.0	.370	8,410	11,520	6,760	9,260	5,970	8,180	5,100	6,980
11.0	.368	8,940	12,210	7,220	9,900	6,410	8,770	5,500	7,540
12.0	.365	9,450	12,900	7,690	10,490	6,840	9,330	5,870	8,020
13.0	.362	9,920	13,520	8,100	11,050	7,210	9,830	6,230	8,460
14.0	.360	10,350	14,060	8,520	11,580	7,600	10,330	6,590	8,960
15.0	.357	10,750	14,610	8,890	12,080	7,970	10,800	6,910	9,370
16.0	.355	11,150	15,100	9,260	12,550	8,320	11,270	7,240	9,810
17.0	.352	11,510	15,550	9,610	13,000	8,670	11,700	7,560	10,210
18.0	.350	11,850	16,000	9,950	13,440	8,960	12,090	7,850	10,590
19.0	.347	12,200	16,410	10,270	13,810	9,260	12,490	8,140	10,960
20.0	.345	12,490	16,800	10,570	14,230	9,570	12,850	8,430	11,330
21.0	.342	12,800	17,180	10,850	14,580	9,860	13,200	8,700	11,680
22.0	.340	13,080	17,540	11,130	14,910	10,130	13,570	8,990	12,000
23.0	.338	13,330	17,830	11,410	15,280	10,380	13,900	9,200	12,310
24.0	.336	13,600	18,170	11,650	15,600	10,640	14,210	9,410	12,580
25.0	.333	13,830	18,450	11,910	15,900	10,880	14,500	9,700	12,930

TABLE 9.—*Live load moments in foot-pounds distributed to edge supports*

Controlling conditions:
H-20 loading.
P=16,000 pounds.
Case I—parallel reinforcement.
Modified formulas.
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

Condition at support		Freely supported		Slabs continuous, 50 percent end restraint		Slabs monolithic, 75 percent end restraint		Fully restrained	
Slab span S, feet	$I=\frac{50}{125+S}$	$M_E=0.01\ PS^2$		$M_E=0.008\ PS^2$		$M_E=0.007\ PS^2$		$M_E=0.005\ PS^2$	
S	I	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$	M_E	$M_E\times(1.0+I)$
2.0	0.394	640	890	510	710	450	625	320	448
2.5	.392	1,000	1,390	800	1,115	700	975	500	695
3.0	.391	1,440	2,000	1,150	1,600	1,005	1,400	720	1,000
3.5	.390	1,960	2,730	1,560	2,180	1,370	1,905	980	1,365
4.0	.388	2,560	3,559	2,050	2,840	1,795	2,490	1,280	1,775
4.5	.386	3,240	4,500	2,585	3,600	2,265	3,150	1,620	2,250
5.0	.385	4,000	5,540	3.200	4,425	2,800	3,875	2,000	2,770
5.5	.383	4,840	6,730	3,875	5,365	3,390	4,690	2,420	3,350
6.0	.382	5,760	7,970	4,610	6,380	4,040	5,585	2,880	3,990
6.5	.380	6,760	9,340	5,410	7,480	4,740	6,540	3,380	4,670
7.0	.379	7,820	10,800	6,250	8,640	5,480	7,560	3,910	5,400
7.5	.377	9,000	12.400	7,200	9,930	6,300	8,670	4,500	6,200
8.0	.376	10,240	14,080	8,200	11,260	7,175	9,850	5,120	7,040
8.5	.375	11,580	15,900	9,259	12,720	8,100	11,130	5.790	7,950
9.0	.373	12,960	17,800	10,380	14,250	9,070	12,460	6,480	8,900
9.5	.372	14,440	19,800	11,550	15,830	10,110	13,880	7,220	9,900
10.0	.370	16,000	21,910	12,820	17,530	11,200	15,330	7,990	10,960
11.0	.368	19,380	26,500	15,500	21,180	13,560	18,580	9,690	13,250
12.0	.365	23,070	31,480	18,440	25,150	16,150	22,030	11,530	15,750
13.0	.362	27,040	36,860	21,620	29,509	18,940	25,780	13,520	18,450
14.0	.360	31,370	42,620	25,120	34,140	21,940	29,870	15,680	21,320
15.0	.357	36.000	48,810	28,780	39,020	25,240	34,180	18,000	24,420
16.0	.355	40,980	55,500	32,750	44,380	28,680	38,880	20,480	27,750
17.0	.352	46,250	62,590	37,000	50,000	32,380	43,750	23,120	31,250
18.0	.350	51.880	70,990	41,500	56,040	36,300	49,000	25,920	35,000
19.0	.347	57,760	77,800	46,200	62,260	40,440	54,500	28,860	38,990
20.0	.345	64,000	86,000	51,200	68,866	41,800	60,260	32,000	43,090
21.0	.342	70,500	94,680	56,450	75,760	49,400	66,300	35,260	47,350
22.0	.340	77,500	103,800	62,000	83,050	54,260	72,750	38,760	51,900
23.0	.338	84,600	113,300	67,759	90,700	59,260	79,300	42,300	56,620
24.0	.336	92,100	123,300	73,709	98,500	61,500	86,300	46,100	61,620
25.0	.333	100,000	133,209	80,009	106,600	70,000	93,300	50,000	66,620

TABLE 10.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]

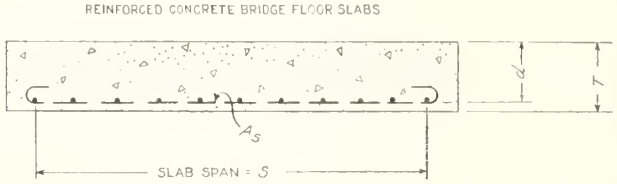


Controlling conditions:
H–15 loading.
Case I—parallel reinforcement.
Freely supported slabs.
All truck lanes.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch and under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

Slab span S	Slab depth T	Bar size [2]	Bar space center-to-center	Dead load moment, $\frac{WS^2}{8}$	$M(1.0+I)$	M_t	A_s	p	k	j	Unit stress	
											f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in*				*Lb./sq. in.*	*Lb./sq. in.*
2	5.75	½ R	5.5	40	2,440	2,480	0.43	0.0079	0.351	0.883	17,400	790
3	6.75	⅝ R	7.5	120	3,480	3,600	.49	.0074	.342	.886	18,100	780
4	7.50	⅝ R	6.5	240	4,430	4,670	.57	.0076	.345	.885	17,750	780
5	8.0	⅝ R	5.5	390	5,300	5,690	.67	.0083	.358	.881	17,200	795
6	8.5	⅝ R	5.0	590	6,080	6,670	.74	.0085	.361	.880	17,100	800
7	9.25	⅝ R	5.0	860	6,810	7,670	.74	.0077	.347	.884	17,700	780
8	9.5	⅝ R	4.5	1,150	7,460	8,610	.82	.0083	.358	.881	17,400	805
9	10.25	⅝ R	4.5	1,550	8,085	9,635	.82	.0076	.345	.885	17,700	780
10	10.75	¾ R	6.0	2,000	8,645	10,645	.88	.0080	.353	.882	17,800	810
11	11.0	¾ R	5.5	2,455	9,165	11,620	.96	.0084	.360	.880	17,350	810
12	12.0	¾ R	5.5	3,150	9,680	12,830	.96	.0076	.345	.885	17,250	760
13	12.0	⅞ R	7.0	3,700	10,140	12,840	1.03	.0082	.356	.881	17,500	800
14	12.5	⅞ R	7.0	4,440	10,550	14,990	1.03	.0078	.349	.884	18,000	800
15	13.0	⅞ R	6.5	5,290	10,960	16,250	1.11	.0080	.353	.882	17,390	790
16	13.5	⅞ R	6.5	6,210	11,330	17,540	1.11	.0077	.347	.884	17,900	795
17	14.0	⅞ R	6.0	7,230	11,670	18,900	1.20	.0089	.353	.882	17,100	780
18	14.5	⅞ R	6.0	8,350	12,000	20,350	1.20	.0077	.347	.884	17,700	785
19	15.0	1 R	7.5	9,620	12,330	21,950	1.26	.0078	.349	.884	17,500	780
20	15.5	1 R	7.5	10,950	12,600	23,550	1.26	.0075	.344	.886	18,100	790
21	16.0	1 R	7.0	12,400	12,880	25,280	1.35	.0077	.347	.884	17,600	785
22	16.5	1 R	7.0	13,980	13,150	27,130	1.35	.0075	.344	.886	18,150	795
23	17.0	1 R	6.5	15,720	13,370	29,090	1.45	.0078	.349	.884	17,600	780
24	17.5	1 R	6.5	17,570	13,630	31,200	1.45	.0076	.345	.885	18,200	800
25	18.0	1 R	6.0	19,530	13,840	33,370	1.57	.0079	.351	.883	17,500	790

[1] See table 2 for reinforcement transverse to the main reinforcement and table 7 for edge support requirements.
[2] In this and subsequent tables R is used to designate round bars.

TABLE 11.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]

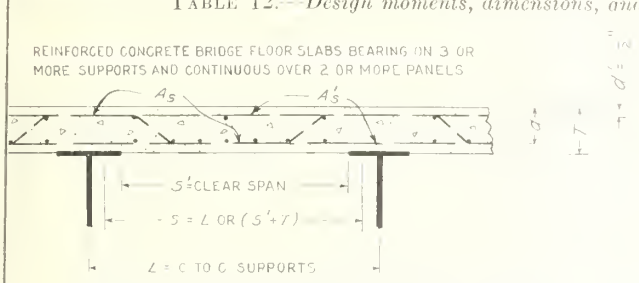


Controlling conditions:
H–15 loading.
Case I—parallel reinforcement.
Freely supported slabs.
All truck lanes.
Paving allowance, 75 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch and under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{8}$	$M(1.0+I)$	M_t	1.	p	k	j	Unit stress	
											f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*				*Lb./sq. in.*	*Lb./sq. in.*
2	5.75	½ R	5.5	70	2,440	2,510	0.43	0.0079	0.351	0.883	17,600	800
3	6.75	⅝ R	7.0	180	3,480	3,660	.53	.0080	.353	.882	17,100	780
4	7.50	⅝ R	6.5	340	4,430	4,770	.57	.0076	.345	.885	18,200	800
5	8.25	⅝ R	5.5	560	5,300	5,860	.67	.0080	.353	.882	17,000	770
6	8.75	⅝ R	5.0	830	6,080	6,910	.74	.0082	.356	.881	17,000	780
7	9.5	⅝ R	5.0	1,190	6,810	8,000	.74	.0075	.344	.886	17,750	775
8	9.75	⅝ R	4.5	1,570	7,460	9,030	.82	.0080	.353	.882	17,600	800
9	10.5	¾ R	6.0	2,085	8,085	10,170	.88	.0082	.356	.881	17,500	800
10	11.0	¾ R	5.5	2,655	8,645	11,300	.96	.0081	.360	.880	16,900	790
11	11.5	¾ R	5.5	3,315	9,165	12,480	.96	.0080	.353	.882	17,700	805
12	12.0	⅞ R	7.0	4,050	9,680	13,730	1.03	.0082	.356	.881	17,300	795
13	12.5	⅞ R	7.0	4,880	10,140	15,020	1.03	.0078	.349	.884	18,000	805
14	13.0	⅞ R	6.5	5,830	10,550	16,380	1.11	.0080	.353	.882	17,500	795
15	13.5	⅞ R	6.5	6,860	10,960	17,820	1.11	.0077	.347	.884	18,200	805
16	14.0	⅞ R	6.0	8,000	11,330	19,330	1.18	.0080	.353	.882	17,550	800
17	14.5	⅞ R	6.0	9,250	11,670	20,920	1.20	.0077	.347	.884	18,200	805
18	15.0	1 R	7.5	10,650	12,000	22,650	1.26	.0078	.349	.884	18,100	805
19	15.5	1 R	7.0	12,140	12,330	24,470	1.35	.0080	.353	.882	17,600	800
20	16.0	1 R	6.5	13,750	12,600	26,350	1.45	.0083	.358	.881	17,100	795
21	16.5	1 R	6.5	15,480	12,880	28,360	1.45	.0081	.354	.882	17,800	810
22	17.0	1 R	6.0	17,390	13,150	30,540	1.57	.0085	.361	.880	17,100	800
23	17.5	1 R	6.0	19,440	13,370	32,810	1.57	.0082	.356	.881	17,700	810
24	18.0	1 R	5.5	21,600	13,630	35,230	1.71	.0086	.363	.879	17,000	805
25	19.0	1 R	5.5	24,460	13,840	38,300	1.71	.0082	.356	.881	17,400	800

[1] See table 2 for reinforcement transverse to the main reinforcement and table 7 for edge support requirements.

TABLE 12.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]


REINFORCED CONCRETE BRIDGE FLOOR SLABS BEARING ON 3 OR MORE SUPPORTS AND CONTINUOUS OVER 2 OR MORE PANELS
A_s
A'_s
S CLEAR SPAN
$L = S + 1$ OR $(S + T)$
L = C TO C SUPPORTS

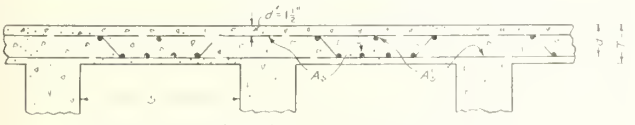
Spans 2 to 3.5 feet, inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\ A_s$, bars bent as shown

Controlling conditions:
H-15 loading.
Case I—parallel reinforcement.
Slabs continuous.
50 percent end restraint.
All truck lanes.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s—Area of tension reinforcement.
A'_s—Area of compression reinforcement.
$d = T - 1.25$ inches for bars $\frac{5}{8}$ inch or under.
$d = T - 1.50$ inches for bars over $\frac{5}{8}$ inch.
All bars to be hooked at ends.

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M\ (1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.25	½ R	6.5	35	1,830	1,865	0.36	0.0075		0.344	0.886	17,500	770
2.5	5.75	½ R	6.0	60	2,230	2,290	.39	.0072	0.333	.338	.887	17,600	750
3.0	6.0	½ R	5.5	90	2,630	2,720	.43	.0075	.316	.338	.880	18,200	770
3.5	6.25	⅝ R	7.5	130	3,020	3,150	.49	.0082	.300	.346	.872	17,700	780
4.0	6.5	⅝ R	7.0	170	3,390	3,560	.53	.0084	.286	.352	.875	17,550	795
4.5	6.75	⅝ R	6.5	220	3,740	3,960	.57	.0086	.273	.353	.872	17,350	795
5.0	7.0	⅝ R	6.0	280	4,090	4,370	.61	.0088	.261	.355	.871	17,100	790
5.5	7.25	⅝ R	6.0	350	4,410	4,760	.61	.0085	.250	.350	.872	17,800	800
6.0	7.5	⅝ R	5.5	430	4,740	5,170	.67	.0089	.240	.355	.871	17,000	780
6.5	7.75	⅝ R	5.5	520	5,040	5,560	.67	.0086	.231	.349	.873	17,600	785
7.0	7.75	⅝ R	5.0	600	5,340	5,940	.74	.0095	.231	.361	.868	17,050	805
7.5	8.0	⅝ R	5.0	700	5,630	6,330	.74	.0091	.222	.355	.871	17,450	805
8.0	8.25	⅝ R	5.0	820	5,920	6,740	.74	.0088	.214	.350	.873	17,900	805
8.5	8.5	⅝ R	4.5	950	6,190	7,140	.82	.0094	.207	.358	.870	16,600	770
9.0	8.5	⅝ R	4.5	1,060	6,450	7,510	.82	.0094	.207	.358	.870	17,400	810
9.5	8.75	⅝ R	4.5	1,210	6,710	7,920	.82	.0091	.200	.353	.872	17,700	805
10.0	9.0	⅝ R	4.5	1,380	6,950	8,330	.82	.0088	.194	.348	.874	18,000	800
11.0	9.5	¾ R	6.0	1,750	7,420	9,170	.88	.0092	.188	.353	.873	17,900	810
12.0	9.75	¾ R	5.5	2,120	7,870	9,990	.96	.0097	.182	.359	.871	17,400	810
13.0	10.25	¾ R	5.5	2,590	8,290	10,880	.96	.0092	.171	.351	.875	17,800	800
14.0	10.5	⅞ R	7.0	3,060	8,690	11,750	1.03	.0095	.167	.356	.874	17,400	800
15.0	11.0	⅞ R	7.0	3,670	9,060	12,730	1.03	.0090	.158	.348	.878	17,800	790
16.0	11.5	⅞ R	7.0	4,330	9,420	13,750	1.03	.0086	.150	.341	.882	18,150	785
17.0	11.5	⅞ R	6.5	4,880	9,760	14,640	1.11	.0092	.150	.350	.879	18,000	805
18.0	12.0	⅞ R	6.0	5,680	10,080	15,760	1.20	.0095	.143	.353	.878	17,100	775
19.0	12.0	1 R	7.5	6,330	10,380	16,710	1.26	.0100	.143	.358	.877	17,300	805
20.0	12.5	1 R	7.5	7,250	10,670	17,920	1.26	.0095	.136	.352	.879	17,700	800
21.0	13.0	1 R	7.5	8,270	10,940	19,210	1.26	.0091	.130	.346	.882	18,100	795
22.0	13.0	1 R	7.0	9,080	11,190	20,270	1.35	.0098	.130	.354	.880	17,800	810
23.0	13.5	1 R	7.0	10,250	11,460	21,710	1.35	.0094	.125	.348	.884	18,200	810
24.0	14.0	1 R	6.5	11,530	11,700	23,230	1.45	.0097	.120	.352	.883	17,400	790
25.0	14.5	1 R	6.5	12,880	11,920	24,800	1.45	.0093	.115	.347	.885	17,800	780

[1] See table 2 for reinforcement transverse to main reinforcement and table 7 for edge support requirements.

TABLE 13.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]


REINFORCED CONCRETE BRIDGE FLOOR SLABS BUILT MONOLITHIC WITH 3 OR MORE SUPPORTS AND CONTINUOUS OVER 2 OR MORE PANELS
$d' = 1\frac{1}{2}$"
A_s
A'_s
S

Spans 2 to 3.5 feet inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet inclusive, $A'_s = 0.5\ A_s$, bars bent as shown.

Controlling conditions:
H-15 loading.
Case I—parallel reinforcement.
Slabs monolithic.
75 percent end restraint.
All truck lanes.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s—Area of tension reinforcement.
A'_s—Area of compression reinforcement.
$d = T - 1.25$ inches for bars $\frac{5}{8}$ inch or under.
$d = T - 1.50$ inches for bars over $\frac{5}{8}$ inch.
All bars to be hooked at ends.

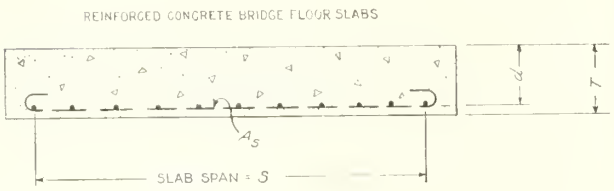
Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in*
2.0	5.0	½ R	7.0	30	1,560	1,590	0.34	0.0075		0.341	0.886	16,900	750
2.5	5.25	½ R	6.0	60	1,920	1,980	.39	.0081		.354	.882	17,300	790
3.0	5.75	½ R	6.0	90	2,270	2,360	.39	.0072	0.333	.338	.887	18,200	775
3.5	6.0	½ R	5.5	120	2,600	2,720	.43	.0075	.316	.338	.880	18,200	770
4.0	6.25	⅝ R	7.5	170	2,920	3,090	.49	.0082	.300	.350	.876	17,300	775
4.5	6.5	⅝ R	7.0	220	3,230	3,450	.53	.0084	.286	.352	.875	17,000	770
5.0	6.75	⅝ R	7.0	280	3,540	3,820	.53	.0080	.273	.345	.876	17,950	790
5.5	7.00	⅝ R	6.5	340	3,830	4,170	.57	.0083	.261	.347	.874	17,500	775
6.0	7.25	⅝ R	6.5	420	4,130	4,550	.57	.0079	.250	.341	.877	18,000	780
6.5	7.25	⅝ R	5.5	490	4,400	4,890	.67	.0093	.250	.361	.868	16,800	790
7.0	7.5	⅝ R	5.5	580	4,670	5,250	.67	.0089	.240	.355	.871	17,300	795
7.5	7.75	⅝ R	5.5	690	4,950	5,610	.67	.0086	.231	.349	.873	17,800	800
8.0	7.75	⅝ R	5.0	780	5,200	5,980	.74	.0095	.231	.361	.868	17,200	810
8.5	8.0	⅝ R	5.0	910	5,450	6,360	.74	.0091	.222	.355	.871	17,550	805
9.0	8.25	⅝ R	5.0	1,040	5,670	6,710	.74	.0088	.214	.350	.873	17,800	800
9.5	8.5	⅝ R	5.0	1,190	5,920	7,110	.74	.0085	.207	.345	.875	18,200	800
10.0	8.5	⅝ R	4.5	1,320	6,140	7,460	.82	.0094	.207	.358	.870	17,300	805

[1] See table 2 for reinforcement transverse to main reinforcement and table 7 for edge support requirements.

TABLE 13.—*Design moments, dimensions, and reinforcement of slabs for the following conditions*—Continued

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
11.0	9.0	$\frac{5}{8}$ R	4.5	1,666	6,580	8,240	0.82	0.0088	0.194	0.348	0.874	17,800	790
12.0	9.5	$\frac{3}{4}$ R	6.0	2,080	7,000	9,080	.88	.0092	.188	.353	.873	17,700	800
13.0	10.0	$\frac{3}{4}$ R	6.0	2,540	7,370	9,910	.88	.0087	.177	.345	.876	18,150	790
14.0	10.25	$\frac{3}{4}$ R	5.5	3,000	7,750	10,750	.96	.0092	.171	.351	.875	17,500	790
15.0	10.5	$\frac{7}{8}$ R	7.0	3,510	8,100	11,610	1.03	.0095	.167	.356	.874	17,200	790
16.0	11.0	$\frac{7}{8}$ R	7.0	4,170	8,460	12,630	1.03	.0090	.158	.348	.878	17,600	785
17.0	11.0	$\frac{7}{8}$ R	6.5	4,710	8,780	13,490	1.11	.0097	.158	.356	.875	17,600	810
18.0	11.5	$\frac{7}{8}$ R	6.5	5,480	9,070	14,550	1.11	.0092	.150	.350	.879	17,900	800
19.0	11.5	1 R	7.5	6,100	9,370	15,470	1.26	.0105	.150	.365	.874	16,850	805
20.0	12.0	1 R	7.5	7,000	9,640	16,640	1.26	.0100	.143	.358	.877	17,200	800
21.0	12.5	1 R	7.5	7,980	9,900	17,880	1.26	.0095	.136	.352	.879	17,600	800
22.0	13.0	1 R	7.5	9,090	10,180	19,270	1.26	.0091	.130	.346	.882	18,100	795
23.0	13.0	1 R	7.0	9,950	10,430	20,380	1.35	.0098	.130	.354	.880	17,900	810
24.0	13.5	1 R	6.5	11,180	10,660	21,840	1.45	.0101	.125	.357	.879	17,200	790
25.0	14.0	1 R	6.5	12,500	10,880	23,380	1.45	.0097	.120	.352	.883	17,600	790

TABLE 14.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]

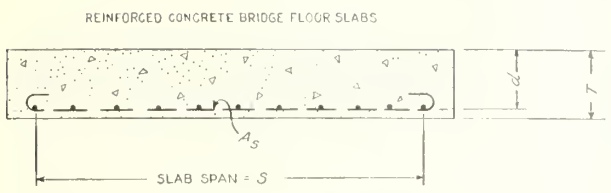


Controlling conditions:
H–20 loading.
Case I—parallel reinforcement.
Freely supported slabs.
All truck lanes.
Paving allowance, 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
$d = T - 1.25$ inches for bars $\frac{5}{8}$ inch and under.
$d = T - 1.50$ inches for bars over $\frac{5}{8}$ inch.
All bars to be hooked at ends.

Slab span S	Slab depth T	Bar size	Bar space, center-to-center	Dead-load moment $\frac{WS^2}{8}$	$M(1.0+I)$	M_t	A_s	p	k	j	Unit stress	
											f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*				*Lb./sq. in.*	*Lb./sq. in.*
2	6.5	$\frac{1}{2}$ R	5.0	50	3,250	3,300	0.47	0.0075	0.344	0.886	18,050	790
3	7.5	$\frac{5}{8}$ R	6.5	140	4,640	4,780	.57	.0076	.345	.885	18,100	800
4	8.25	$\frac{5}{8}$ R	5.5	260	5,900	6,160	.67	.0080	.353	.882	17,900	810
5	9.0	$\frac{5}{8}$ R	5.0	430	7,060	7,490	.74	.0079	.351	.883	17,800	800
6	9.75	$\frac{5}{8}$ R	4.5	660	8,110	8,770	.82	.0080	.353	.882	17,200	780
7	10.5	$\frac{5}{8}$ R	4.5	950	9,080	1,030	.82	.0074	.342	.886	17,900	775
8	11.0	$\frac{3}{4}$ R	6.0	1,300	9,940	11,240	.88	.0077	.347	.884	18,200	810
9	11.5	$\frac{3}{4}$ R	5.5	1,710	10,770	12,480	.96	.0080	.353	.882	17,650	800
10	12.0	$\frac{7}{8}$ R	7.0	2,190	11,520	13,710	1.03	.0082	.356	.881	17,300	790
11	12.5	$\frac{7}{8}$ R	7.0	2,740	12,210	14,950	1.03	.0078	.349	.884	17,900	800
12	13.0	$\frac{7}{8}$ R	6.5	3,380	12,900	16,280	1.11	.0080	.353	.882	17,400	790
13	13.5	$\frac{7}{8}$ R	6.5	4,100	13,520	17,620	1.11	.0077	.347	.884	17,950	800
14	14.0	$\frac{7}{8}$ R	6.0	4,900	14,060	18,960	1.20	.0080	.353	.882	17,150	780
15	14.5	$\frac{7}{8}$ R	6.0	5,790	14,610	20,400	1.20	.0077	.347	.884	17,700	790
16	14.5	$\frac{7}{8}$ R	5.5	6,590	15,100	21,690	1.31	.0084	.360	.880	17,300	810
17	15.0	$\frac{7}{8}$ R	5.5	7,670	15,550	23,220	1.31	.0081	.354	.882	17,800	810
18	15.5	1 R	7.0	8,860	16,000	24,860	1.35	.0080	.353	.882	17,900	810
19	16.0	1 R	6.5	10,150	16,440	26,590	1.45	.0083	.358	.881	17,200	800
20	16.5	1 R	6.5	11,540	16,800	28,340	1.45	.0080	.353	.882	17,750	810
21	17.0	1 R	6.0	13,100	17,180	30,280	1.57	.0081	.360	.880	17,000	795
22	17.5	1 R	6.0	14,770	17,540	32,310	1.57	.0082	.356	.881	17,500	805
23	18.0	1 R	6.0	16,530	17,830	34,360	1.57	.0079	.351	.883	18,000	810
24	18.5	1 R	5.5	18,430	18,170	36,600	1.71	.0084	.360	.880	17,200	805
25	19.0	1 R	5.5	20,500	18,450	38,950	1.71	.0082	.356	.881	17,700	810

[1] See table 2 for reinforcement transverse to main reinforcement and table 9 for edge support requirements.

TABLE 15.—*Design moments, dimensions, and reinforcement for the following conditions*[1]

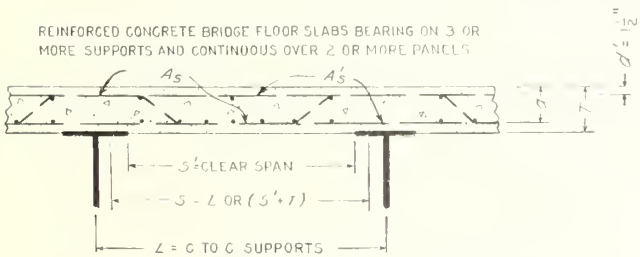


Controlling conditions:
H–20 loading.
Case I—parallel reinforcement.
Freely supported slabs.
All truck lanes.
Paving allowance, 75 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch and under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

Slab span, S	Slab depth, T	Bar size	Bar space, center-to-center	Dead-load moment $\frac{W'S^2}{8}$	$M(1.0+I)$	M_t	A_s	p	k	j	Unit stress f_s	Unit stress f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*				*Lb. sq. in.*	*Lb./sq. in.*
2	6.5	½ R	5.0	80	3,250	3,330	0.47	0.0075	0.344	0.886	18,200	795
3	7.5	⅝ R	6.0	190	4,640	4,830	.61	.0082	.356	.881	17,250	790
4	8.5	⅝ R	5.5	360	5,900	6,260	.67	.0077	.347	.884	17,500	775
5	9.25	⅝ R	5.0	600	7,060	7,660	.74	.0077	.347	.884	17,550	780
6	9.75	⅝ R	4.5	890	8,110	9,000	.82	.0080	.353	.882	17,600	800
7	10.5	¾ R	6.0	1,260	9,080	10,340	.88	.0082	.356	.881	17,700	810
8	11.0	¾ R	5.5	1,700	9,940	11,640	.96	.0084	.360	.880	17,300	810
9	12.0	¾ R	5.5	2,280	10,770	13,050	.96	.0076	.345	.885	17,500	775
10	12.5	⅞ R	7.0	2,890	11,520	14,410	1.03	.0078	.349	.884	17,250	770
11	13.0	⅞ R	7.0	3,600	12,210	15,810	1.03	.0075	.344	.886	18,050	785
12	13.0	⅞ R	6.0	4,280	12,930	17,180	1.20	.0087	.365	.878	17,000	810
13	13.5	⅞ R	5.5	5,150	13,520	18,670	1.31	.0091	.371	.876	16,200	800
14	14.0	⅞ R	5.5	6,130	14,060	20,190	1.31	.0087	.365	.878	16,800	805
15	14.5	⅞ R	5.5	7,200	14,610	21,810	1.31	.0084	.360	.880	17,400	810
16	15.0	1 R	7.0	8,100	15,100	23,500	1.35	.0083	.358	.881	17,550	810
17	15.5	1 R	6.5	9,740	15,550	25,260	1.45	.0086	.363	.879	17,000	805
18	16.5	1 R	6.5	11,380	16,000	27,380	1.45	.0080	.353	.882	17,100	780
19	17.0	1 R	6.5	12,980	16,440	29,420	1.45	.0078	.349	.884	17,800	795
20	17.5	1 R	6.0	14,700	16,800	31,500	1.57	.0082	.356	.881	17,100	785
21	18.0	1 R	6.0	16,540	17,180	33,720	1.57	.0079	.351	.883	17,650	795
22	18.5	1 R	5.5	18,500	17,540	36,040	1.71	.0084	.360	.880	16,900	790
23	19.0	1 R	5.5	20,670	17,830	38,500	1.71	.0082	.356	.881	17,500	800
24	19.5	1 R	5.5	22,960	18,170	41,130	1.71	.0079	.351	.883	18,100	810
25	20.0	[2]1 S	6.5	25,400	18,450	43,850	1.85	.0083	358	881	17,500	810

[1] See table 2 for reinforcement transverse to main reinforcement and table 9 for edge support requirements.
[2] S is used here to denote square bars.

TABLE 16.—*Design moments, dimensions, and reinforcement of slabs for the following conditions*[1]


REINFORCED CONCRETE BRIDGE FLOOR SLABS BEARING ON 3 OR MORE SUPPORTS AND CONTINUOUS OVER 2 OR MORE PANELS
A_s
A'_s
d' = 1/2 d
d
T
S' = CLEAR SPAN
S - L OR (S' + t)
L = C TO C SUPPORTS

Spans 2 to 3.5 feet, inclusive, $A'_s = .1 A_s$, bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\ A_s$, bars bent as shown.

Controlling conditions:
H 20 loading.
Case I—parallel reinforcement.
Slabs continuous.
50 percent end restraint.
All truck lanes.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
A'_s = Area of compression reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch or under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

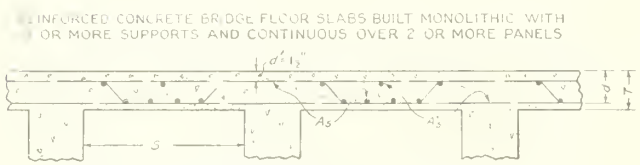
Slab span S	Slab depth T	Bar size	Bar space, center-to-center	Dead-load moment $\frac{W'S^2}{10}$	$M\ (1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress f_s	Unit stress f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.75	½ R	5.5	10	2,440	2,480	0.13	0.0080	0.333	0.349	0.878	17,600	785
2.5	6.25	⅝ R	8.0	60	2,970	3,030	.46	.0077	.300	.338	.877	18,000	770
3.0	6.5	⅝ R	7.0	100	3,510	3,610	.53	.0084	.286	.346	.868	17,900	790
3.5	6.75	⅝ R	6.0	130	4,030	4,160	.61	.0093	.273	.355	.862	17,200	785
4.0	7.25	⅝ R	6.0	180	4,520	4,700	.61	.0085	.250	.350	.872	17,700	790
4.5	7.5	⅝ R	5.5	240	4,980	5,220	.67	.0089	.240	.355	.871	17,200	790
5.0	7.75	⅝ R	5.5	310	5,450	5,760	.67	.0086	.231	.349	.873	18,200	810
5.5	8.0	⅝ R	5.0	380	5,880	6,260	.74	.0091	.222	.355	.871	17,250	795
6.0	8.25	⅝ R	5.0	460	6,320	6,780	.74	.0088	.214	.350	.873	18,000	810
6.5	8.5	⅝ R	4.5	550	6,720	7,270	.82	.0094	.207	.358	.870	16,900	785
7.0	8.75	⅝ R	4.5	660	7,120	7,780	.82	.0091	.200	.353	.872	17,400	790
7.5	9.0	⅝ R	4.5	780	7,510	8,290	.82	.0088	.194	.348	.874	17,900	800
8.0	9.5	⅝ R	4.5	920	7,900	8,820	.82	.0083	.182	.340	.877	17,800	765
8.5	9.75	¾ R	6.0	1,060	8,250	9,310	.88	.0089	.182	.348	.875	17,500	780
9.0	9.75	¾ R	5.5	1,190	8,600	9,790	.96	.0097	.182	.359	.871	17,000	790
9.5	10.0	¾ R	5.5	1,360	8,940	10,300	.96	.0094	.177	.354	.874	17,300	790
10.0	10.25	¾ R	5.5	1,530	9,260	10,790	.96	.0092	.171	.351	.875	17,600	790
11.0	10.5	⅞ R	7.0	1,890	9,900	11,790	1.03	.0095	.167	.356	.874	17,500	800
12.0	11.0	⅞ R	7.0	2,340	10,490	12,830	1.03	.0090	.158	.348	.878	17,900	795
13.0	11.5	⅞ R	6.5	2,860	11,050	13,910	1.11	.0092	.150	.350	.879	17,100	770
14.0	11.5	⅞ R	6.0	3,310	11,580	14,890	1.20	.0100	.150	.359	.875	17,000	795
15.0	12.0	⅞ R	6.0	3,940	12,080	16,020	1.20	.0095	.143	.353	.878	17,300	785
16.0	12.0	1R	7.5	4,480	12,550	17,030	1.26	.0100	.143	.358	.877	17,600	810
17.0	12.5	1R	7.5	5,230	13,000	18,230	1.26	.0095	.136	.352	.879	18,000	810

[1] See table 2 for reinforcement transverse to main reinforcement and table 9 for edge support requirement.

TABLE 16.—*Design moments, dimensions, and reinforcement of slabs for the following conditions*—Continued

Slab span S	Slab depth T	Bar size	Bar space, center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
18.0	13.0	1 R	7.0	6,070	13,440	19,510	1.35	0.0098	0.130	0.354	0.880	17,200	785
19.0	13.0	1 R	6.5	6,770	13,840	20,610	1.45	.0105	.130	.363	.878	16,900	800
20.0	13.5	1 R	6.5	7,720	14,230	21,950	1.45	.0101	.125	.357	.879	17,200	795
21.0	14.0	1 R	6.5	8,820	14,580	23,400	1.45	.0097	.120	.352	.883	17,600	790
22.0	14.0	1 R	6.0	9,680	14,910	24,590	1.57	.0105	.120	.361	.880	17,100	805
23.0	14.5	1 R	6.0	10,900	15,280	26,180	1.57	.0101	.115	.356	.881	17,500	800
24.0	15.0	1 R	6.0	12,240	15,600	27,840	1.57	.0097	.111	.351	.884	17,800	800
25.0	15.5	1 R	6.0	13,680	15,900	29,580	1.57	.0093	.107	.346	.887	18,200	805

TABLE 17.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]



Spans 2 to 3.5 feet inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet inclusive, $A'_s = 0.5\,A_s$, bars bent as shown.

Controlling conditions:
H–20 loading.
Case I—parallel reinforcement.
Slabs monolithic.
75 percent end restraint.
All truck lanes.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
A'_s = Area of compression reinforcement.
$d = T - 1.25$ inches for bars ⅞ inch or under.
$d = T - 1.50$ inches for bars over ⅞ inch.
All bars to be hooked at ends.

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq.-in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.5	½ R	6.0	40	2,080	2,120	0.39	0.0076	-------	0.345	0.885	17,300	765
2.5	5.75	½ R	5.0	60	2,560	2,620	.47	.0087	0.333	.359	.872	17,000	795
3.0	6.25	⅝ R	7.5	90	3,030	3,120	.49	.0082	.300	.346	.872	17,500	770
3.5	6.5	⅝ R	7.0	130	3,470	3,600	.53	.0084	.286	.346	.868	17,900	790
4.0	6.75	⅝ R	6.5	170	3,890	4,060	.57	.0086	.273	.353	.872	17,800	810
4.5	7.25	⅝ R	6.5	240	4,310	4,550	.57	.0079	.250	.341	.877	18,000	780
5.0	7.25	⅝ R	5.5	290	4,720	5,010	.67	.0093	.250	361	.868	17,300	810
5.5	7.5	⅝ R	5.0	360	5,110	5,470	74	.0098	.240	.367	.866	16,500	790
6.0	7.75	⅝ R	5.0	440	5,500	5,940	74	.0095	.231	.361	.868	17,050	805
6.5	8.0	⅝ R	5.0	530	5,860	6,390	.71	.0091	.222	.355	.871	17,700	810
7.0	8.25	⅝ R	4.5	630	6,220	6,850	.82	.0097	.214	.363	.868	16,500	785
7.5	8.5	⅝ R	4.5	740	6,600	7,340	.82	.0094	.207	.358	.870	17,100	790
8.0	8.75	⅝ R	4.5	860	6,930	7,790	.82	.0091	.200	.353	.872	17,500	790
8.5	9.0	⅝ R	4.5	990	7,270	8,260	.82	.0088	.194	.348	.874	17,900	795
9.0	9.25	⅝ R	4.5	1,140	7,560	8,700	.82	.0085	.188	.343	.877	18,200	795
9.5	9.5	¾ R	6.0	1,300	7,890	9,190	.88	.0092	.188	.353	.873	17,900	810
10.0	9.75	¾ R	6.0	1,470	8,180	9,650	.88	.0089	.182	.348	.875	18,200	810
11.0	10.0	¾ R	5.5	1,820	8,770	10,590	.96	.0091	.177	.354	.874	17,800	810
12.0	10.5	⅞ R	7.0	2,250	9,330	11,580	1.03	.0095	167	.356	.874	17,100	785
13.0	11.0	⅞ R	7.0	2,750	9,830	12,580	1.03	.0090	.158	.348	.878	17,500	780
14.0	11.5	⅞ R	7.0	3,310	10,330	13,640	1.03	.0086	.150	.341	.882	18,050	780
15.0	11.5	⅞ R	6.5	3,800	10,800	14,600	1.11	.0092	.150	.350	.879	18,000	805
16.0	12.0	⅞ R	6.0	4,480	11,270	15,750	1.20	.0095	.143	.353	.878	17,050	780
17.0	12.0	1 R	7.5	5,060	11,700	16,760	1.26	0100	.143	.358	.877	17,400	805
18.0	12.5	1 R	7.5	5,870	12,090	17,960	1.26	.0095	.136	.352	.879	17,700	800
19.0	13.0	1 R	7.5	6,770	12,490	19,260	1.26	.0091	.130	.346	.882	18,100	795
20.0	13.0	1 R	7.0	7,500	12,850	20,350	1.35	.0098	.130	.354	.880	17,950	810
21.0	13.5	1 R	7.0	8,550	13,200	21,750	1.35	.0094	.125	.348	.884	18,200	810
22.0	14.0	1 R	6.5	9,680	13,570	23,250	1.45	.0097	.120	.352	.883	17,500	790
23.0	14.0	1 R	6.0	10,590	13,900	24,490	1.57	.0105	.120	.361	.880	17,100	790
24.0	14.5	1 R	6.0	11,870	14,210	26,080	1.57	.0101	.115	.356	.881	17,400	800
25.0	15.0	1 R	6.0	13,280	14,500	27,780	1.57	.0097	.111	.351	.884	17,800	800

[1] See table 2 for reinforcement transverse to main reinforcement and table 9 for edge support requirements.

Case II.—Modified formulas will also be applied in the design of reinforced concrete bridge floor slabs with main reinforcement transverse to the direction of traffic.

The 1935 specifications of the American Association of State Highway Officials will be used, modified as follows:

Live-load moments computed by modified formulas as shown in table 18.

Unit stresses: $f'_c = 3{,}000$ pounds per square inch; $f_s = 18{,}000$ pounds per square inch; $f_c = 800$ pounds per square inch; $n = 12$.

Live load: *H*–15 and *H*–20 truck train loading.

TABLE 18.—*Summary of modified formulas for case II*

Condition of slab at support	Spans (feet)	Live-load moments (in foot-pounds per foot width of slab)		Dead-load moment
		Exterior spans	Interior spans	
Freely supported.	0 to 4	$M=\frac{P\sqrt{S}}{9.64}$	$M=\frac{P\sqrt{S}}{9.64}$	$\frac{WS^2}{8}$
	4 to 10	$M=\frac{PS}{2.32S+10}$	$M=\frac{PS}{1.32S+14}$	
Slabs continuous 50 percent end restraint.	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.035P$	$M=\frac{P\sqrt{S}}{9.64}-0.035P$	$\frac{WS^2}{10}$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.035P$	$M=\frac{PS}{1.32S+14}-0.035P$	
Slabs monolithic 75 percent end restraint.	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.0525P$	$M=\frac{P\sqrt{S}}{9.64}-0.0525P$	$\frac{WS^2}{10}$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.0525P$	$M=\frac{PS}{1.32S+14}-0.0525P$	
Fully restrained.	0 to 4	$M=\frac{P\sqrt{S}}{9.64}-0.07P$	$M=\frac{P\sqrt{S}}{9.64}-0.07P$	$\frac{WS^2}{12}$
	4 to 10	$M=\frac{PS}{2.32S+10}-0.07P$	$M=\frac{PS}{1.32S+14}-0.07P$	

Symbols are defined as follows:

S=Effective design span in feet.
W=Dead load per square foot of slab.
P=Wheel load=12,000 pounds for H–15 and 16,000 pounds for H–20 loading.
I=Impact factor=$\frac{50}{125+S}$
M=Maximum live-load moment.
$M(1.0+I)$=Live load+impact moment.
M_t=Design moment=Dead-load moment+$M(1.0+I)$.

All moments are in foot-pounds per foot width of slab.

For values of live-load moments in foot-pounds per foot width of slab by above formulas see tables 19 and 20.

For design moments, dimensions, and reinforcement of slabs for case II see tables 21 to 24.

TABLE 19.—*Live-load moments in foot-pounds per foot width of slab*

Controlling conditions:
H–15 loading.
P=12,000 pounds.
Case II—transverse reinforcement.
Modified formulas
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

INTERIOR SPANS

Condition at support		Freely supported		Slabs continuous 50 percent end restraint		Slabs monolithic 75 percent end restraint		Fully restrained	
Slab span S in feet	$I=\frac{50}{125+S}$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$	M	$M\times(1.0+I)$
2.0	0.394	1,760	2,450	1,340	1,870	1,130	1,575	922	1,285
2.5	.392	1,970	2,740	1,550	2,160	1,340	1,865	1,129	1,571
3.0	.391	2,160	3,005	1,740	2,420	1,530	2,130	1,318	1,835
3.5	.390	2,340	3,250	1,920	2,670	1,710	2,380	1,490	2,070
4.0	.388	2,490	3,460	2,070	2,875	1,860	2,580	1,650	2,290
4.5	.386	2,710	3,755	2,290	3,175	2,080	2,880	1,873	2,595
5.0	.385	2,910	4,030	2,490	3,450	2,280	3,160	2,072	2,870
5.5	.383	3,110	4,300	2,690	3,720	2,480	3,430	2,270	3,140
6.0	.382	3,280	4,530	2,860	3,950	2,650	3,660	2,440	3,370
6.5	.380	3,160	4,770	3,040	4,200	2,830	3,900	2,618	3,610
7.0	.379	3,610	4,980	3,190	4,400	2,980	4,110	2,770	3,820
7.5	.377	3,770	5,185	3,350	4,610	3,140	4,320	2,925	4,030
8.0	.376	3,910	5,380	3,490	4,800	3,280	4,510	3,070	4,220
8.5	.375	4,050	5,560	3,630	4,990	3,420	4,700	3,205	4,410
9.0	.373	4,170	5,720	3,750	5,150	3,540	4,860	3,340	4,590
9.5	.372	4,300	5,900	3,880	5,320	3,670	5,040	3,450	4,730
10.0	.370	4,410	6,040	3,990	5,470	3,780	5,180	3,570	4,890

EXTERIOR SPANS

2.0	0.394	1,760	2,450	1,340	1,870	1,130	1,575	922	1,285
2.5	.392	1,970	2,740	1,550	2,160	1,310	1,865	1,129	1,571
3.0	.391	2,160	3,005	1,740	2,420	1,530	2,130	1,318	1,835
3.5	.390	2,340	3,250	1,920	2,670	1,710	2,380	1,490	2,070
4.0	.388	2,490	3,460	2,070	2,875	1,860	2,580	1,650	2,290
4.5	.386	2,640	3,660	2,220	3,080	2,010	2,785	1,800	2,495
5.0	.385	2,780	3,850	2,360	3,270	2,150	2,980	1,940	2,685
5.5	.383	2,900	4,010	2,480	3,430	2,270	3,140	2,060	2,850
6.0	.382	3,010	4,160	2,590	3,580	2,380	3,290	2,170	3,000
6.5	.380	3,110	4,290	2,690	3,710	2,480	3,420	2,270	3,135
7.0	.379	3,200	4,410	2,780	3,830	2,570	3,540	2,360	3,260
7.5	.377	3,290	4,530	2,870	3,950	2,660	3,660	2,450	3,370
8.0	.376	3,360	4,620	2,940	4,050	2,730	3,760	2,520	3,470
8.5	.375	3,440	4,730	3,020	4,150	2,810	3,860	2,590	3,560
9.0	.373	3,500	4,810	3,080	4,230	2,870	3,940	2,660	3,650
9.5	.372	3,560	4,880	3,140	4,310	2,930	4,020	2,720	3,740
10.0	.370	3,620	4,960	3,200	4,380	2,990	4,100	2,770	3,790

TABLE 20.—*Live-load moments in foot-pounds per foot width of slab*

Controlling conditions:
H–20 loading.
P=16,000 pounds.
Case II—transverse reinforcement.
Modified formulas.
Freely supported.
Slabs continuous.
Slabs monolithic.
Fully restrained.

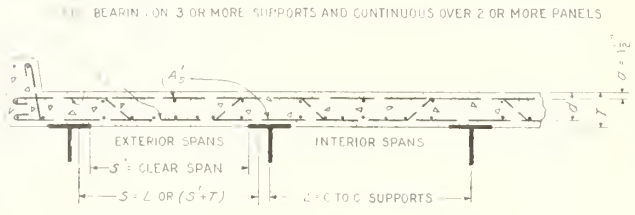
INTERIOR SPANS

Condition at support		Freely supported		Slabs continuous 50 percent end restraint		Slabs monolithic 75 percent end restraint		Fully restrained	
Slab span S in feet	$I=\frac{50}{125+S}$	M	$M(1.0+I)$	M	$M(1.0+I)$	M	$M(1.0+I)$	M	$M(1.0+I)$
2.0	0.394	2,340	3,260	1,780	2,480	1,500	2,090	1,230	1,710
2.5	.392	2,620	3,650	2,060	2,870	1,780	2,480	1,505	2,100
3.0	.391	2,880	4,010	2,320	3,230	2,040	2,840	1,760	2,450
3.5	.390	3,110	4,320	2,550	3,540	2,270	3,150	1,985	2,760
4.0	.388	3,320	4,610	2,760	3,830	2,480	3,440	2,200	3,050
4.5	.386	3,610	5,000	3,050	4,230	2,770	3,840	2,197	3,460
5.0	.385	3,880	5,370	3,320	4,600	3,040	4,210	2,765	3,830
5.5	.383	4,150	5,740	3,590	4,970	3,310	4,580	3,028	4,190
6.0	.382	4,380	6,050	3,820	5,280	3,540	4,890	3,250	4,490
6.5	.380	4,610	6,360	4,050	5,590	3,770	5,200	3,490	4,820
7.0	.379	1,820	6,650	4,260	5,880	3,980	5,490	3,690	5,090
7.5	.377	5,030	6,930	4,470	6,160	4,190	5,770	3,900	5,370
8.0	.376	5,210	7,170	4,650	6,400	4,370	6,020	4,090	5,630
8.5	.375	5,400	7,420	4,840	6,660	4,560	6,270	4,270	5,870
9.0	.373	5,560	7,640	5,000	6,870	4,720	6,480	4,450	6,110
9.5	.372	5,730	7,860	5,170	7,100	4,890	6,710	4,600	6,310
10.0	.370	5,880	8,060	5,320	7,290	5,010	6,900	4,760	6,520

EXTERIOR SPANS

2.0	0.394	2,340	3,260	1,780	2,480	1,500	2,090	1,230	1,710
2.5	.392	2,620	3,650	2,060	2,870	1,780	2,480	1,505	2,100
3.0	.391	2,880	4,010	2,320	3,230	2,010	2,810	1,760	2,450
3.5	.390	3,110	4,320	2,550	3,540	2,270	3,150	1,985	2,760
4.0	.388	3,320	4,610	2,760	3,830	2,480	3,440	2,200	3,050
4.5	.386	3,520	4,880	2,960	4,100	2,680	3,710	2,400	3,330
5.0	.385	3,710	5,140	3,150	4,360	2,870	3,970	2,587	3,580
5.5	.383	3,870	5,350	3,310	4,580	3,030	4,190	2,747	3,800
6.0	.382	4,020	5,560	3,460	4,780	3,180	4,390	2,890	3,990
6.5	.380	4,150	5,730	3,590	1,960	3,310	4,570	3,026	1,180
7.0	.379	4,270	5,890	3,710	5,120	3,430	4,730	3,148	4,340
7.5	.377	4,380	6,030	3,820	5,260	3,540	4,870	3,267	4,500
8.0	.376	4,480	6,160	3,920	5,400	3,640	5,010	3,360	4,630
8.5	.375	4,580	6,300	4,020	5,530	3,740	5,140	3,450	4,740
9.0	.373	4,670	6,420	4,110	5,640	3,830	5,260	3,548	4,870
9.5	.372	4,750	6,520	4,190	5,750	3,910	5,360	3,625	4,970
10.0	.370	4,820	6,600	4,260	5,840	3,980	5,460	3,690	5,060

TABLE 21.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]


BEARING ON 3 OR MORE SUPPORTS AND CONTINUOUS OVER 2 OR MORE PANELS
EXTERIOR SPANS
INTERIOR SPANS
S = CLEAR SPAN
L = C TO C SUPPORTS

Spans 2 to 3.5 feet, inclusive, $A'_s = A_s$ bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\ A_s$, bars bent as shown.

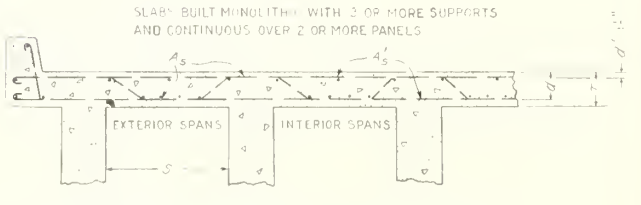
Controlling conditions:
H–15 loading.
Case II—transverse reinforcement.
Slabs continuous.
50 percent end restraint.
Paving allowance, 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
A'_s = Area of compression reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch or under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

INTERIOR OR EXTERIOR SPANS

Slab span, S	Slab depth, T	Bar size	Bar space, center-to-center	Dead load moment, $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.25	½ R	6.5	40	1,870	1,910	0.36	0.0075	.....	0.344	0.886	17,950	785
2.5	5.5	½ R	6.0	60	2,160	2,220	.39	.0076	.....	.345	.885	18,159	800
3.0	5.75	½ R	5.5	90	2,420	2,510	.43	.0080	0.333	.349	.878	17,700	790
3.5	6.00	½ R	8.0	120	2,670	2,790	.46	.0082	.316	.349	.874	17,590	780
4.0	6.25	½ R	8.0	160	2,880	3,040	.46	.0077	.300	.342	.880	18,000	780
EXTERIOR SPANS													
4.5	6.25	⅝ R	7.9	210	3,080	3,290	0.53	0.0088	0.300	0.359	0.871	17,100	800
5.0	6.50	⅝ R	7.0	270	3,270	3,540	.53	.0084	.286	.352	.875	17,450	790
5.5	6.50	⅝ R	6.5	320	3,430	3,750	.57	.0090	.286	.361	.870	17,250	810
6.0	6.75	⅝ R	6.5	390	3,580	3,970	.57	.0086	.273	.353	.872	17,400	790
6.5	7.00	⅝ R	6.5	480	3,710	4,190	.57	.0079	.261	.347	.874	17,550	775
7.0	7.00	⅝ R	6.0	550	3,830	4,380	.61	.0088	.261	.355	.871	17,200	790
7.5	7.25	⅝ R	6.0	650	3,950	4,600	.61	.0085	.250	.350	.872	17,300	780
8.0	7.25	⅝ R	6.0	740	4,050	4,790	.61	.0085	.250	.350	.872	18,000	805
8.5	7.25	⅝ R	5.5	830	4,150	4,980	.67	.0093	.250	.361	.868	17,160	805
9.0	7.50	⅝ R	5.5	960	4,230	5,190	.67	.0089	.240	.355	.871	17,100	780
9.5	7.50	⅝ R	5.5	1,080	4,310	5,390	.67	.0089	.240	.355	.871	17,750	810
10.0	7.75	⅝ R	5.5	1,220	4,380	5,600	.67	.0086	.231	.349	.873	17,700	790
INTERIOR SPANS													
4.5	6.50	⅝ R	7.5	220	3,180	3,400	0.49	0.0078	0.286	0.343	0.879	18,050	785
5.0	6.50	⅝ R	6.5	270	3,450	3,720	.57	.0090	.286	.361	.870	17,200	810
5.5	6.75	⅝ R	6.5	330	3,720	4,050	.57	.0086	.273	.353	.872	17,750	810
6.0	7.00	⅝ R	6.5	400	3,950	4,350	.57	.0083	.261	.347	.874	18,200	805
6.5	7.25	⅝ R	6.0	490	4,200	4,690	.61	.0085	.250	.350	.872	17,600	790
7.0	7.25	⅝ R	5.5	570	4,400	4,970	.67	.0093	.250	.361	.868	17,100	805
7.5	7.50	⅝ R	5.5	670	4,610	5,280	.67	.0089	.240	.355	.871	17,400	800
8.0	7.75	⅝ R	5.5	780	4,800	5,580	.67	.0086	.231	.349	.873	17,600	785
8.5	7.75	⅝ R	5.0	880	4,990	5,870	.74	.0095	.231	.361	.868	16,900	795
9.0	8.00	⅝ R	5.0	1,010	5,150	6,160	.74	.0091	.222	.355	.871	17,000	780
9.5	8.25	⅝ R	5.0	1,160	5,320	6,480	.74	.0088	.214	.350	.873	17,200	770
10.0	8.25	⅝ R	5.0	1,280	5,470	6,750	.74	.0088	.214	.350	.873	17,900	800

[1] See tabulation, p. 155, for reinforcement transverse to main reinforcement.

TABLE 22.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]



Spans 2 to 3.5 feet, inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\ A_s$, bars bent as shown.

Controlling conditions:
H–15 loading.
Case II—transverse reinforcement.
Slabs monolithic.
75 percent end restraint.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
A'_s = Area of compression reinforcement.
$d = T - 1.25$ inches for bars ⅝ inch or under.
$d = T - 1.50$ inches for bars over ⅝ inch.
All bars to be hooked at ends.

INTERIOR OR EXTERIOR SPANS

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.00	½ R	7.0	35	1,575	1,610	0.34	0.0075		0.344	0.886	17,100	750
2.5	5.25	½ R	6.5	60	1,865	1,925	.36	.0075	.....	.344	.886	18,000	790
3.0	5.50	½ R	6.0	90	2,130	2,220	.39	.0076	.....	.345	.885	18,100	800
3.5	5.75	½ R	5.5	120	2,380	2,500	.43	.0080	0.333	.349	.878	17,600	785
4.0	6.00	½ R	5.5	160	2,580	2,740	.43	.0075	.316	.342	.882	18,200	790

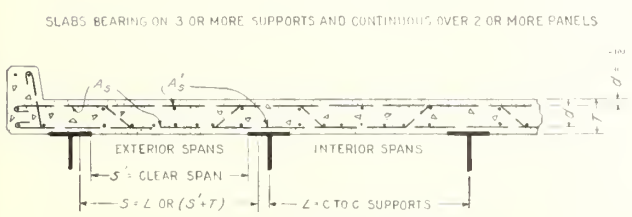
[1] See tabulation page 155 for reinforcement transverse to main reinforcement.

DESIGN 22.—*Design moments, dimensions, and reinforcement of slabs for the following conditions*—Continued

EXTERIOR SPANS

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
4.5	6.00	$\frac{5}{8}$ R	7.5	205	2,785	2,990	0.49	0.0086	0.316	0.358	0.874	17,600	810
5.0	6.25	$\frac{5}{8}$ R	7.5	260	2,980	3,240	.49	.0082	.300	.350	.876	18,100	810
5.5	6.50	$\frac{5}{8}$ R	7.0	320	3,140	3,460	.53	.0084	.286	.352	.875	17,100	775
6.0	6.50	$\frac{5}{8}$ R	6.5	380	3,290	3,670	.57	.0090	.286	.361	.870	16,950	800
6.5	6.75	$\frac{5}{8}$ R	6.5	460	3,420	3,880	.57	.0086	.273	.353	.872	17,000	775
7.0	6.75	$\frac{5}{8}$ R	6.5	530	3,540	4,070	.57	.0086	.273	.353	.872	17,850	810
7.5	7.00	$\frac{5}{8}$ R	6.5	630	3,660	4,290	.57	.0083	.261	.347	.874	18,000	795
8.0	7.25	$\frac{5}{8}$ R	6.5	740	3,760	4,500	.57	.0079	.250	.341	.877	18,000	780
8.5	7.25	$\frac{5}{8}$ R	6.0	840	3,860	4,700	.61	.0085	.250	.350	.872	17,700	795
9.0	7.25	$\frac{5}{8}$ R	5.5	940	3,940	4,880	.67	.0093	.250	.360	.868	16,800	790
9.5	7.50	$\frac{5}{8}$ R	5.5	1,070	4,020	5,090	.67	.0089	.240	.355	.871	16,750	770
10.0	7.50	$\frac{5}{8}$ R	5.5	1,190	4,100	5,290	.67	.0089	.240	.355	.871	17,400	800
						INTERIOR SPANS							
4.5	6.25	$\frac{5}{8}$ R	7.5	210	2,880	3,090	0.49	0.0082	0.300	0.350	0.876	17,300	775
5.0	6.50	$\frac{5}{8}$ R	7.5	260	3,160	3,420	.49	.0078	.286	.343	.879	18,150	790
5.5	6.50	$\frac{5}{8}$ R	6.5	320	3,430	3,750	.57	.0090	.286	.361	.870	17,300	810
6.0	6.75	$\frac{5}{8}$ R	6.5	390	3,660	4,050	.57	.0086	.273	.353	.872	17,800	810
6.5	7.00	$\frac{5}{8}$ R	6.0	480	3,900	4,380	.61	.0088	.261	.355	.871	17,500	790
7.0	7.25	$\frac{5}{8}$ R	6.0	570	4,110	4,680	.61	.0085	.250	.350	.872	17,600	790
7.5	7.25	$\frac{5}{8}$ R	5.5	650	4,320	4,970	.67	.0093	.250	.361	.868	17,100	805
8.0	7.50	$\frac{5}{8}$ R	5.5	760	4,510	5,270	.67	.0089	.240	.355	.871	17,300	795
8.5	7.75	$\frac{5}{8}$ R	5.5	880	4,700	5,580	.67	.0086	.231	.349	.873	17,600	790
9.0	7.75	$\frac{5}{8}$ R	5.0	990	4,860	5,850	.74	.0095	.231	.361	.868	16,800	790
9.5	8.00	$\frac{5}{8}$ R	5.0	1,130	5,040	6,170	.74	.0091	.222	.355	.871	17,000	780
10.0	8.00	$\frac{5}{8}$ R	5.0	1,250	5,180	6,430	.74	.0091	.222	.355	.871	17,800	810

TABLE 23.—*Design moments, dimensions, and reinforcement of slabs for the following conditions*[1]


SLABS BEARING ON 3 OR MORE SUPPORTS AND CONTINUOUS OVER 2 OR MORE PANELS
A_s
A'_s
EXTERIOR SPANS
INTERIOR SPANS
S = CLEAR SPAN
S + L OR (S'+T)
L = C TO C SUPPORTS

Spans 2 to 3.5 feet, inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\ A_s$, bars bent as shown.

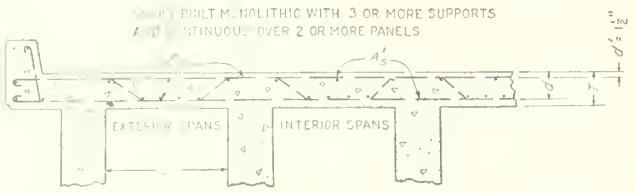
Controlling conditions:
H–20 loading.
Case II—transverse reinforcement.
Slabs continuous.
50 percent end restraint.
Paving allowance 25 pounds per square foot.
Modified formulas.
A_s = Area of tension reinforcement.
A'_s = Area of compression reinforcement.
$d = T - 1.25$ inches for bars $\frac{5}{8}$ inch or under.
$d = T - 1.50$ inches for bars over $\frac{5}{8}$ inch.
All bars to be hooked at ends.

INTERIOR OR EXTERIOR SPANS

Slab span S	Slab depth T	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.75	$\frac{1}{2}$R	5.5	40	2,480	2,520	0.43	0.0080	0.333	0.349	0.878	17,800	795
2.5	6.00	$\frac{1}{2}$R	5.0	70	2,870	2,940	.47	.0082	.316	.349	.874	18,100	810
3.0	6.25	$\frac{5}{8}$R	7.0	90	3,230	3,320	.53	.0088	.300	.354	.867	17,350	790
3.5	6.50	$\frac{5}{8}$R	6.5	130	3,540	3,670	.57	.0090	.286	.353	.864	17,000	775
4.0	6.75	$\frac{5}{8}$R	6.5	170	3,830	4,000	.57	.0086	.273	.353	.872	17,550	800
						EXTERIOR SPANS							
4.5	7.00	$\frac{5}{8}$ R	6.5	230	4,100	4,330	0.57	0.0083	0.261	0.347	0.874	18,100	800
5.0	7.25	$\frac{5}{8}$ R	6.0	290	4,360	4,650	.61	.0085	.250	.350	.872	17,500	785
5.5	7.25	$\frac{5}{8}$ R	5.5	350	4,580	4,930	.67	.0093	.250	.361	.868	16,950	800
6.0	7.50	$\frac{5}{8}$ R	5.5	430	4,780	5,210	.67	.0089	.240	.355	.871	17,150	785
6.5	7.75	$\frac{5}{8}$ R	5.5	520	4,960	5,480	.67	.0086	.231	.349	.873	17,250	770
7.0	7.75	$\frac{5}{8}$ R	5.5	600	5,120	5,720	.67	.0086	.231	.349	.873	18,050	810
7.5	7.75	$\frac{5}{8}$ R	5.0	690	5,260	5,950	.74	.0095	.231	.361	.868	17,100	805
8.0	8.00	$\frac{5}{8}$ R	5.0	800	5,400	6,200	.74	.0091	.222	.355	.871	17,100	785
8.5	8.00	$\frac{5}{8}$ R	5.0	900	5,530	6,430	.74	.0091	.222	.355	.871	17,700	810
9.0	8.25	$\frac{5}{8}$ R	5.0	1,040	5,640	6,680	.74	.0088	.214	.350	.873	17,750	795
9.5	8.50	$\frac{5}{8}$ R	5.0	1,180	5,750	6,930	.74	.0085	.207	.345	.875	17,700	780
10.0	8.50	$\frac{5}{8}$ R	4.5	1,310	5,840	7,150	.82	.0094	.207	.358	.870	16,600	770
						INTERIOR SPANS							
4.5	7.00	$\frac{5}{8}$ R	6.0	230	4,230	4,460	0.61	0.0089	0.261	0.357	0.871	17,550	810
5.0	7.25	$\frac{5}{8}$ R	5.5	290	4,600	4,890	.67	.0093	.250	.361	.868	16,800	790
5.5	7.50	$\frac{5}{8}$ R	5.5	360	4,970	5,330	.67	.0089	.240	.355	.871	17,500	800
6.0	7.75	$\frac{5}{8}$ R	5.5	440	5,280	5,720	.67	.0086	.231	.349	.873	18,050	805
6.5	8.00	$\frac{5}{8}$ R	5.0	530	5,590	6,120	.74	.0091	.222	.355	.871	16,900	775
7.0	8.25	$\frac{5}{8}$ R	5.0	630	5,880	6,510	.74	.0088	.214	.350	.873	17,250	775
7.5	8.25	$\frac{5}{8}$ R	4.5	720	6,160	6,880	.82	.0097	.214	.363	.868	16,600	790
8.0	8.50	$\frac{5}{8}$ R	4.5	840	6,400	7,240	.82	.0094	.207	.358	.870	16,800	780
8.5	8.75	$\frac{5}{8}$ R	4.5	970	6,660	7,630	.82	.0091	.200	.353	.872	17,050	775
9.0	8.75	$\frac{5}{8}$ R	4.5	1,090	6,870	7,960	.82	.0091	.200	.353	.872	17,800	810
9.5	9.00	$\frac{5}{8}$ R	4.5	1,240	7,100	8,340	.82	.0088	.194	.348	.875	17,950	800
10.0	9.25	$\frac{5}{8}$ R	4.5	1,410	7,290	8,700	.82	.0085	.188	.343	.877	18,100	790

[1] See tabulation p. 155 for reinforcement transverse to main reinforcement.

TABLE 24.—*Design moments, dimensions, and reinforcement of slabs for the following conditions* [1]



Spans 2 to 3.5 feet, inclusive, $A'_s = A_s$, bars not bent.
Spans 4 to 10 feet, inclusive, $A'_s = 0.5\,A_s$, bars bent as shown.

Controlling conditions:
- *H*–20 loading.
- Case II—transverse reinforcement.
- Slabs monolithic.
- 75 percent end restraint.
- Paving allowance, 25 pounds per square foot.
- Modified formulas.
- A_s = Area of tension reinforcement.
- A'_s = Area of compression reinforcement.
- $d = T - 1.25$ inches for bars ⅞ inch or under.
- $d = T - 1.50$ inches for bars over ⅞ inch.
- All bars to be hooked at ends.

Slab span *S*	Slab depth *T*	Bar size	Bar space center-to-center	Dead load moment $\frac{WS^2}{10}$	$M(1.0+I)$	M_t	A_s	p	$\frac{d'}{d}$	k	j	Unit stress	
												f_s	f_c
Feet	*Inches*	*Inch*	*Inches*	*Ft.-lb.*	*Ft.-lb.*	*Ft.-lb.*	*Sq. in.*					*Lb./sq. in.*	*Lb./sq. in.*
2.0	5.50	½ R	6.0	40	2,090	2,130	0.39	0.0076	----------	0.345	0.885	17,400	770
2.5	5.75	½ R	5.5	60	2,480	2,540	.43	.0080	0.333	.349	.878	17,950	800
3.0	6.00	½ R	5.0	90	2,840	2,930	.47	.0082	.316	.349	.874	18,000	805
3.5	6.25	⅝ R	7.0	130	3,150	3,280	.53	.0088	.300	.354	.865	17,200	790
4.0	6.50	⅝ R	7.0	170	3,440	3,610	.53	.0084	.286	.352	.875	17,800	810
EXTERIOR SPANS													
4.5	6.75	⅝ R	6.5	220	3,710	3,930	0.57	0.0086	0.273	0.353	0.872	17,250	785
5.0	7.00	⅝ R	6.5	280	3,970	4,250	.57	.0083	.261	.347	.874	17,800	790
5.5	7.25	⅝ R	6.5	350	4,190	4,540	.57	.0079	.250	.341	.877	18,000	780
6.0	7.25	⅝ R	6.0	420	4,390	4,810	.61	.0085	.250	.350	.872	18,100	810
6.5	7.50	⅝ R	6.0	500	4,570	5,070	.61	.0081	.240	.344	.875	18,200	795
7.0	7.50	⅝ R	5.5	580	4,730	5,310	.67	.0089	.240	.355	.871	17,500	800
7.5	7.75	⅝ R	5.5	690	4,870	5,560	.67	.0086	.231	.349	.873	17,550	785
8.0	7.75	⅝ R	5.0	780	5,010	5,790	.74	.0095	.231	.361	.868	16,600	785
8.5	7.75	⅝ R	5.0	880	5,140	6,020	.74	.0095	.231	.361	.868	17,300	810
9.0	8.00	⅝ R	5.0	1,010	5,260	6,270	.74	.0091	.222	.355	.871	17,300	795
9.5	8.25	⅝ R	5.0	1,160	5,360	6,520	.74	.0088	.214	.350	.873	17,360	780
10.0	8.25	⅝ R	5.0	1,280	5,460	6,740	.74	.0088	.214	.350	.873	17,850	800
INTERIOR SPANS													
4.5	6.75	⅝ R	6.5	220	3,840	4,060	0.57	0.0086	0.273	0.353	0.872	17,800	810
5.0	7.00	⅝ R	6.0	280	4,210	4,490	.61	.0088	.261	.356	.871	17,600	810
5.5	7.25	⅝ R	5.5	350	4,580	4,930	.67	.0093	.250	.361	.868	17,000	800
6.0	7.50	⅝ R	5.5	430	4,890	5,320	.67	.0089	.240	.355	.871	17,500	800
6.5	7.75	⅝ R	5.5	520	5,200	5,720	.67	.0086	.231	.349	.873	18,050	805
7.0	8.00	⅝ R	5.0	610	5,490	6,100	.74	.0091	.222	.355	.871	16,850	770
7.5	8.25	⅝ R	5.0	720	5,770	6,490	.74	.0088	.214	.350	.873	17,250	775
8.0	8.50	⅝ R	5.0	840	6,020	6,860	.74	.0085	.207	.345	.875	17,500	770
8.5	8.50	⅞ R	4.5	950	6,270	7,220	.82	.0094	.207	.358	.870	16,750	780
9.0	8.50	⅝ R	4.5	1,060	6,480	7,540	.82	.0094	.207	.358	.870	17,500	810
9.5	8.75	⅝ R	4.5	1,210	6,710	7,920	.82	.0091	.200	.353	.872	17,700	800
10.0	9.00	⅞ R	4.5	1,380	6,900	8,280	.82	.0088	.194	.348	.874	17,900	800

[1] See tabulation p. 155 for reinforcement transverse to main reinforcement.

Part 4.—EXAMPLES OF SLAB DESIGN BY MODIFIED FORMULAS

The following examples of slab designs are given in order to illustrate the application of the modified formulas and tables advanced in this analysis.

Example 1.—Required: To design a bridge floor slab of uniform thickness, providing a 24-foot clear roadway for *H*–15 loading, with main reinforcement transverse to the direction of traffic.

The size and spacing of the supporting stringers will depend on the bridge span and for this example are assumed to be as shown by figure 13. The stringer flanges in figure 13 are assumed to be 12 inches wide.

The effective design span S is shown by the figure at the top of table 21 to be the clear span S' plus total slab depth T, or $S=S'+T$. Trial slab depths for the above spans are shown in table 21 for 50 percent end restraint to be 6¾ inches for exterior spans and 7 inches for interior spans.

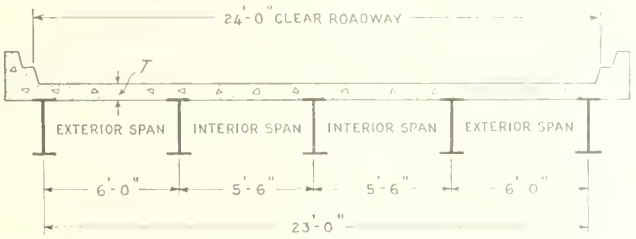


FIGURE 13.—BRIDGE SPAN AND SPACING OF STRINGERS ASSUMED FOR EXAMPLE 1.

Using the above trial slab depths and solving for the effective design span S results in the following slabs:

Exterior spans: S=(6 feet)−12 inches+6¾ inches=5 feet 6¾ inches.

Slab required from table 21: T=6½ inches and A_s=⅝-inch diameter bars at 6.5-inch centers=0.57 square inches.

Interior spans: S=(5 feet 6 inches)−12 inches+7 inches=5 feet 1 inch.

Slab required from table 21: T=6½ inches and A_s=⅝-inch diameter bars at 6.5-inch centers=0.57 square inches.

Reinforcement required in the bottom of the slab in the direction of the y-axis, from the tabulation on page 155:

Middle half of slab span: A_s=65 percent of 0.57=0.37 square inches=⅝-inch diameter bars at 10-inch centers.

Outer quarters of slab span: A_s=45 percent of 0.57=0.27 square inches=⅝-inch diameter bars at 13½-inch centers.

Where stringers are of uniform spacing and the slab is of uniform thickness the design of the slab is determined by the requirements for interior spans.

In the design of bridge floor slabs for case II, where the slabs are monolithic with the supporting beams, the procedure is similar to that followed in example 1 except that the clear span between supports is used as the design span of slab.

Example 2.—Required: To design a bridge floor slab of uniform thickness providing a 24-foot clear roadway for *H*–15 loading, with main reinforcement parallel to the direction of traffic.

It is assumed that the slab is supported on at least three I-beams, the beams having flanges 12 inches wide and spaced at 15-foot centers.

Under the conditions assumed above, it will be found that the effective design span is 15 feet and for this span, table 12, 50 percent end restraint shows the required slab thickness, T=11 inches and the main reinforcement A_s=1.03 square inches=⅞-inch diameter bars at 7-inch centers.

Reinforcement required in bottom of slab in direction of y-axis, from table 2:

Middle half of span: 35 percent of 1.03 square inches = 0.36 square inches = ⅝-inch diameter bars at 10-inch centers.

Outer quarters of span: 25 percent of 1.03 square inches = 0.26 square inches ⅝-inch diameter bars at 14-inch centers.

Table 7 shows that for a 15-foot span, with 50 percent end restraint, the edge supports must provide for live load plus impact moments $M_E(1.0+I)$=29,280 foot-pounds. To this live-load moment must be added the dead-load moment of edge support and the design then completed as for an ordinary concrete beam.

It will be found that generally a properly proportioned curb and that section of the floor slab directly below the curb, with proper reinforcement, will together provide an edge support of adequate strength.

HIGHWAY RESEARCH BOARD TO MEET SOON

The Seventeenth Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., Tuesday, November 30 to Friday, December 3, 1937. Reports on highway research investigations will be presented. This year the formal meeting of the Board will be interspersed with open meetings for informal discussion of pertinent topics. A program of reports is to be announced in the near future.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF SEPTEMBER 30, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 1,904,861	$ 952,430	81.2	$ 3,892,490	$ 1,946,240	180.2	$ 4,948,510
Arizona	$ 493,737	$ 352,427	26.3	1,674,618	1,207,502	62.9	387,064	246,827	13.5	1,496,273
Arkansas	951,061	951,061	49.4	2,987,580	2,981,461	206.9	264,896	264,568	13.4	2,201,562
California	4,130,314	2,176,652	89.3	6,115,326	3,339,231	99.4	1,591,714	850,109	32.3	2,545,078
Colorado	2,120,146	1,184,994	80.3	1,374,756	759,648	41.0	297,633	151,302	1.5	2,275,521
Connecticut	433,578	214,586	4.6	508,530	246,433	5.3	72,610	36,300	.2	1,513,584
Delaware	145,000	72,500	5.3	367,312	183,567	14.2	243,905	119,934	14.8	1,107,687
Florida	134,650	67,320	6.6	2,886,732	1,443,366	66.8	397,670	198,835	6.5	2,701,380
Georgia	1,296,635	648,307	74.8	3,960,707	1,980,349	184.8	1,586,786	793,393	62.1	5,240,785
Idaho	1,355,247	809,466	118.3	1,388,134	829,769	86.7	255,051	151,329	16.5	1,175,308
Illinois	3,269,564	1,600,307	95.9	9,444,533	4,701,925	298.6	4,942,023	2,464,300	77.2	2,194,856
Indiana	1,958,389	979,305	33.5	5,036,618	2,518,188	147.8	1,441,072	720,756	28.4	2,110,500
Iowa	3,368,984	1,545,380	105.4	6,538,400	2,911,416	192.5	1,344,990	604,627	48.7	1,023,707
Kansas	1,571,591	758,540	97.9	4,696,994	2,348,398	174.1	1,867,742	933,869	98.2	3,290,233
Kentucky	476,246	238,123	11.1	4,110,237	2,005,119	117.6	1,667,898	833,949	96.6	2,580,372
Louisiana	213,312	102,519	5.9	6,753,832	1,500,885	48.6	5,293,838	836,455	12.5	1,740,479
Maine	946,670	473,335	28.6	2,162,366	1,081,183	54.2	947,940	473,970	18.3	250,269
Maryland	180,456	90,228	2.1	1,585,150	792,542	26.3	842,318	421,159	12.2	1,790,879
Massachusetts	387,641	193,820	3.2	4,574,608	2,287,304	19.3	111,392	55,695		2,551,513
Michigan	3,623,700	1,811,850	97.1	6,720,410	3,360,705	163.0	2,745,879	1,294,714	46.6	249,218
Minnesota	2,474,120	1,228,319	123.5	4,768,060	2,313,416	177.9	2,224,570	779,760	89.6	1,794,472
Mississippi	472,300	236,150	22.7	4,305,190	2,152,430	209.8	2,209,100	1,102,750	86.4	3,141,846
Missouri	4,030,612	1,973,986	203.8	6,827,637	3,290,264	295.9	3,726,529	1,566,545	102.7	2,005,637
Montana	2,413,790	1,353,022	179.0	2,407,497	1,322,078	139.7	725,714	408,031	27.1	2,355,546
Nebraska	1,148,631	574,315	108.7	5,023,735	2,531,230	510.1	2,457,254	567,243	59.8	2,458,943
Nevada	913,415	783,466	63.3	1,759,200	1,521,487	57.6	116,897	101,370	15.0	963,072
New Hampshire	143,483	71,379	1.9	666,718	329,127	11.2				1,032,363
New Jersey				2,497,729	1,171,249	22.2	18,550	9,275		2,299,806
New Mexico	1,813,138	1,115,241	135.8	2,507,364	1,628,618	123.8	946,882	577,521	60.9	323,532
New York	4,449,714	2,004,726	67.3	19,063,681	9,091,106	325.9	3,743,980	1,834,430	59.8	1,040,960
North Carolina	1,682,619	841,250	114.9	4,898,694	2,307,247	304.4	2,203,818	980,139	67.1	2,976,231
North Dakota	656,200	656,200	125.4	765,740	745,230	118.6	398,912	398,912	56.3	3,921,891
Ohio	1,096,282	541,498	17.5	9,586,100	4,700,014	106.1	2,312,460	1,156,230	26.3	5,688,182
Oklahoma	1,356,190	712,093	50.4	3,391,256	1,759,570	143.0	1,787,641	937,843	83.5	3,264,447
Oregon	1,827,166	1,110,135	85.7	2,919,130	1,737,784	84.5	549,140	315,006	25.7	1,120,809
Pennsylvania	4,467,081	2,225,990	60.0	11,104,713	5,538,585	156.1	2,415,464	1,188,621	43.5	3,804,702
Rhode Island	536,250	266,814	5.7	1,095,487	547,734	13.7	235,380	117,690	1.7	772,483
South Carolina	1,716,535	727,200	137.4	4,658,992	1,885,758	228.1	1,921,048	743,335	72.8	1,500,411
South Dakota	1,222,122	702,796	82.4	2,102,240	1,167,856	230.0	522,978	289,160	72.5	3,387,450
Tennessee	735,218	360,664	31.3	1,567,844	783,922	44.6	995,620	497,810	37.8	4,877,275
Texas	5,055,162	2,525,140	398.6	9,844,999	4,904,555	508.9	3,926,376	1,947,043	192.2	6,954,421
Utah	300,225	215,894	21.7	1,564,033	1,112,421	145.2	234,270	163,350	14.5	1,095,388
Vermont	664,181	322,733	18.1	1,196,520	531,579	31.5	618,288	249,409	16.7	149,111
Virginia	1,460,880	730,440	80.7	2,695,538	1,308,679	71.5	1,460,792	712,161	54.2	2,166,446
Washington	1,096,527	576,500	57.5	2,902,159	1,517,879	47.8	392,378	171,300	4.0	1,223,763
West Virginia	528,936	264,468	15.4	1,514,074	756,130	43.3	642,490	439,445	15.9	2,227,236
Wisconsin	5,035,283	2,362,245	136.5	6,459,255	3,136,120	204.0	409,575	203,400	18.9	1,433,196
Wyoming	1,471,229	896,956	142.6	1,752,454	1,063,914	215.0	437,010	265,990	36.6	325,249
District of Columbia										
Hawaii	452,077	224,945	8.0	501,659	244,927	9.9	443,871	203,860	6.9	1,155,475
Puerto Rico							305,890	151,070	4.1	473,930
TOTALS	76,276,289	39,875,285	3,431.4	195,139,402	98,532,330	6,671.5	68,577,788	31,477,030	2,132.2	108,921,987

U. S. GOVERNMENT PRINTING OFFICE: 1937

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF SEPTEMBER 30, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,689,902	$ 3,652,352	134.1	$ 439,718	$ 439,718	10.6				$ 59,045
Arizona	2,569,841	3,120,812	2,448,834	194.8	43,485	43,485					77,522
Arkansas	3,352,061	3,099,822	3,080,734	338.8	225,432	224,123	21.4				47,204
California	7,747,928	7,545,686	7,194,711	257.5	572,929	515,656	6.4				37,561
Colorado	3,395,263	2,505,903	2,459,381	101.0	89,597	89,596	6.0	$ 8,200	$ 8,200		838,086
Connecticut	1,418,709	940,745	889,894	14.5	463,940	422,110	8.0	124,130	64,435	0.2	42,270
Delaware	900,310	605,917	580,185	48.9	278,892	278,892	17.9				41,233
Florida	2,597,144	2,560,805	2,498,097	98.2	63,157	63,157	.9				35,890
Georgia	4,988,967	1,123,540	1,105,384	74.7	2,214,430	2,067,050	115.3	1,276,064	1,155,064	44.7	661,469
Idaho	2,222,747	2,252,832	2,151,909	185.6	49,205	49,131	.3				21,707
Illinois	8,694,009	7,973,079	7,764,820	449.6	826,743	826,743	38.4				102,446
Indiana	4,941,255	4,651,000	4,410,858	196.7	545,867	504,767	41.7				25,629
Iowa	4,991,664	5,092,889	4,755,400	525.5	209,697	202,094	2.8	30,622	29,470	.3	4,700
Kansas	4,994,975	4,301,433	4,260,654	347.3	693,749	651,577	34.5	9,794	9,794	8.8	72,951
Kentucky	3,726,271	3,328,797	3,184,708	348.3	459,839	459,839	9.6	57,000	57,000	.6	24,724
Louisiana	2,890,429	2,424,814	2,167,717	157.0	660,493	599,816	10.7	112,700	74,701	10.4	48,194
Maine	1,676,799	1,487,034	1,474,500	68.9	179,746	177,147	7.2	25,152	25,152		
Maryland	1,750,738	612,216	605,440	20.8	606,836	606,836	14.2	193,134	159,302	6.4	379,160
Massachusetts	3,262,885	398,411	398,411	4.2	2,609,126	2,218,356	14.2	1,142,956	574,593	.8	71,525
Michigan	6,301,414	6,505,842	5,940,287	287.1	303,037	301,921	4.8	44,500	33,882	.3	25,324
Minnesota	5,277,145	5,995,874	4,987,078	888.4	476,734	269,135	14.2	33,901	20,932	.6	
Mississippi	3,457,552	2,799,627	2,795,809	194.5	574,998	573,958	41.1	8,900	8,900	.6	78,886
Missouri	6,012,652	5,076,063	4,967,856	772.2	1,024,997	904,435	5.5	34,390	32,294		108,067
Montana	3,676,416	3,434,392	3,420,696	192.8	278,930	237,591	9.9	8,462	8,462		9,668
Nebraska	3,870,739	3,218,185	3,117,016	339.0	658,455	658,451	31.4	83,806	83,806	2.1	11,467
Nevada	2,243,074	2,289,662	2,209,009	110.1	33,646	33,646					419
New Hampshire	945,225	871,786	840,112	37.7	67,367	67,367	2.5				37,746
New Jersey	3,129,805	1,060,345	1,057,687	16.8	2,038,819	2,025,664	18.6	6,110	6,110		40,344
New Mexico	2,871,397	2,620,712	2,615,546	196.8	233,950	233,950	16.8	14,681	12,196		9,705
New York	11,046,377	10,520,284	10,079,031	167.4	375,889	375,889	2.7	121,300	81,300	.5	510,157
North Carolina	4,720,173	4,004,967	3,934,114	258.6	727,882	727,281	32.0	39,700	39,700	.5	19,077
North Dakota	2,867,245	2,434,477	2,455,696	378.9	112,955	112,899	1.2	278,840	278,840	36.0	19,810
Ohio	7,670,815	5,873,697	5,792,828	231.2	1,749,607	1,709,557	62.1	104,430	104,430	3.3	64,000
Oklahoma	4,580,670	4,193,456	4,080,165	387.6	454,326	454,326	17.6	77,400	35,307	3.8	10,871
Oregon	3,038,642	3,127,168	2,861,796	161.0	134,854	134,854	3.6				41,992
Pennsylvania	9,347,797	3,221,797	3,035,775	155.8	5,741,708	5,327,526	105.0	1,036,513	840,527	25.0	143,970
Rhode Island	989,208	1,109,360	986,896	18.8	2,240	2,240					72
South Carolina	2,702,012	2,262,013	2,112,597	224.6	627,419	545,752	24.9	9,664	9,664	1.5	33,999
South Dakota	2,976,454	2,420,331	2,416,851	450.7	543,257	543,257	53.9				16,346
Tennessee	4,192,460	2,882,289	2,855,497	118.1	1,201,513	1,201,513	30.4	127,960	127,960	2.4	7,489
Texas	11,989,350	12,606,777	11,543,301	1,105.8	382,509	285,584	5.9	159,161	143,575	13.3	16,891
Utah	2,067,154	2,044,610	1,843,023	204.2	196,775	196,775	4.5				27,356
Vermont	924,306	1,016,181	883,048	21.9	49,422	36,400	1.3				4,858
Virginia	3,652,667	3,304,947	3,231,420	1,009.9	217,653	214,356	23.8	16,980	15,878	.4	191,013
Washington	3,026,161	3,315,317	2,900,575	163.4	85,272	85,272	.9	25,465	20,845	.7	19,468
West Virginia	2,231,412	1,364,296	1,354,648	58.7	972,482	836,584	36.5	47,560	40,180	5.4	
Wisconsin	4,823,884	5,250,257	4,725,599	343.4	94,268	93,900	.3				4,385
Wyoming	2,219,155	2,185,929	2,180,273	152.4	33,287	33,287					5,595
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	623,701	605,700	8.9	334,743	265,689	8.5				54,644
TOTALS	195,000,000	166,349,979	157,863,414	12,231.9	30,961,875	28,929,152	920.0	5,259,475	4,102,499	168.6	4,104,935

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF SEPTEMBER 30, 1937

STATE	APPORTIONMENT	COMPLETED		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,037,998	$ 3,020,448	42	1	12	$ 882,072	$ 882,072	7			$ 185,100	$ 108,800	4			$ 23,297
Arizona	1,256,099	1,113,686	1,079,056	13			185,468	163,059	2		3	6,403	6,403			2	7,581
Arkansas	3,574,060	2,754,944	2,748,164	48	6	3	634,348	633,513	7		5	160,617	160,347	1		25	32,036
California	7,486,362	6,603,168	6,384,827	38	8		1,082,369	1,079,589	9			10,000	10,000	5			11,946
Colorado	2,631,567	1,462,997	1,400,633	19	1		881,188	881,188	9	1		377,010	339,301	3		21	10,444
Connecticut	1,712,684	297,379	297,379	2	1	1	1,341,801	1,320,440	8								94,865
Delaware	418,239	130,000	130,000	1			277,993	277,993	2								10,246
Florida	2,827,883	2,181,925	2,178,863	26	5	2	342,524	341,918	5			132,520	132,520			73	174,582
Georgia	4,895,949	252,327	250,835	7	4	1	1,369,471	1,369,471	28	7	4	808,650	808,650	10	2	47	2,466,993
Idaho	1,674,479	1,261,332	1,253,853	19	2	9	396,821	396,731	4		8	4,921	4,921			4	18,974
Illinois	10,307,184	7,753,806	7,706,056	60	7		2,368,150	2,368,150	13	1		226,000	226,000		2		6,978
Indiana	5,111,096	3,582,911	3,424,333	32	12		1,625,286	1,625,286	10		162						61,477
Iowa	5,600,679	4,128,087	4,041,559	90	9	7	1,480,254	1,477,631	17			80,407	68,690			1	12,799
Kansas	5,246,258	4,000,835	3,939,399	54		5	1,273,119	1,253,704	4	1		108,004	15,000	2			38,155
Kentucky	3,672,387	1,195,369	1,187,927	16	4		2,148,221	1,858,489	8	7		573,365	573,365	4	1		52,606
Louisiana	3,213,467	1,260,353	1,260,353	13	1		1,096,237	1,096,219	11	1		842,856	775,566	7	1		81,330
Maine	1,426,861	1,069,189	1,066,911	19		2	375,052	327,380	1	2		4,380	4,380			2	28,190
Maryland	2,061,751	501,739	501,739	3	3	16	759,670	759,670	5	1	3	590,362	517,893	4	1	11	282,449
Massachusetts	4,210,833	2,035,116	2,035,031	17	4		1,709,751	1,709,751	9			249,991	249,991	1			216,060
Michigan	6,765,197	5,983,269	5,767,480	43	6		969,077	898,608	1	2		43,500	43,500		1		55,609
Minnesota	5,395,441	4,190,471	4,095,559	77	13	39	1,299,905	1,288,752	9	1	11						11,131
Mississippi	3,241,475	1,896,581	1,896,417	42	4	2	819,408	819,408	13	3	6	77,000	77,000	2		13	448,650
Missouri	6,142,153	2,337,883	2,278,385	24	1		3,976,810	3,842,459	25		1	1,650	1,650			1	19,660
Montana	2,722,327	2,621,142	2,534,020	37	7		245,576	185,931	1	1							2,376
Nebraska	3,556,441	2,659,232	2,622,097	73	3	18	713,266	713,266	8		1	219,670	194,632	5		5	26,446
Nevada	887,260	885,084	857,417	8	3		13,308	13,308				3,630	3,630			5	12,905
New Hampshire	822,484	543,076	543,031	6	4		292,094	267,945	4		2	11,508	11,508			5	
New Jersey	3,983,826	1,368,744	1,357,699	12	3	1	2,527,814	2,527,814	12	6		44,730	44,730		1		53,583
New Mexico	1,725,286	1,647,440	1,637,999	18	1		82,384	82,384	1								4,903
New York	13,577,189	9,429,245	9,154,721	30	34		4,151,280	4,151,280	15	15		94,000	94,000				177,189
North Carolina	4,823,958	3,087,524	3,082,898	42	18		1,223,939	1,208,439	16		108	431,270	431,270	4			101,350
North Dakota	3,207,473	2,227,259	2,223,856	45	4		983,486	983,486	11	1							131
Ohio	8,439,397	1,790,539	1,681,388	12	3	2	6,090,644	5,627,411	34	5	1	963,579	932,874	13		10	198,225
Oklahoma	5,004,711	3,448,816	3,441,297	54	7	4	1,476,294	1,343,194	11	4	3	202,170	202,170		1	34	18,050
Oregon	2,334,204	2,239,057	2,154,389	15	6	2	164,842	161,937	2								17,878
Pennsylvania	11,433,513	5,051,439	4,649,856	56	14	9	6,874,415	6,396,082	30	9		150,473	150,473	5			287,202
Rhode Island	699,691	653,760	652,694	4	2		44,314	44,314		1							2,683
South Carolina	3,059,956	1,361,019	1,342,028	28	8	4	1,205,870	1,198,439	17	7	24	211,477	211,477	1	1	20	308,011
South Dakota	3,249,086	2,040,762	2,040,075	40	5	50	991,229	991,229	25	1	3	269,350	210,393	17			7,390
Tennessee	3,903,979	1,164,324	1,154,641	23	3	23	2,238,340	2,238,340	22		3	250,230	250,230	1	1	13	260,768
Texas	10,855,982	9,409,434	9,402,105	123	14	64	786,800	786,797	4	1	5	372,155	360,311		1	80	306,769
Utah	1,230,763	930,600	929,195	12	1	1	290,423	290,423	5								11,145
Vermont	729,357	560,407	533,839	7	6	17	204,133	187,274	3	1	3						8,744
Virginia	3,774,287	2,347,895	2,238,024	39	15	13	1,375,798	1,369,173	11	4		159,260	157,246	2	2	11	9,843
Washington	3,095,041	2,459,830	2,429,602	21	11	9	640,152	639,726	2	1		4,562	4,562			2	21,151
West Virginia	2,677,937	266,614	266,614	3		4	2,308,846	2,306,878	20	4	1	27,947	27,947	3		2	76,498
Wisconsin	5,022,683	4,054,169	4,018,147	37	5		973,618	973,618		2		68,667	29,367			19	1,550
Wyoming	1,360,841	1,164,879	1,164,770	12			189,365	189,365	2								6,706
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	293,667	292,776	3			226,162	158,370	2								2,556
TOTALS	196,000,000	123,155,101	120,761,189	1468	269	320	63,609,387	61,707,904	465	90	357	7,967,414	7,440,797	94	15	406	6,090,110

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 9 NOVEMBER 1937

CHANNEL CHANGE ADJACENT TO FOREST HIGHWAY IN OREGON

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

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BUREAU OF PUBLIC ROADS

Volume 18, No. 9 November 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CHANNEL CHANGES ON FOREST HIGHWAYS

Reported by H. D. FARMER, Senior Highway Engineer, and A. B. LEWELLEN, Chief Engineering Inspector-Superintendent, District 1, Bureau of Public Roads

DURING the past 6 or 8 years numerous forest highways in Washington, Oregon, and Montana, have been designed and constructed that involved channel changes to improve the alinement. Prior to this time alinement standards had been lower and there was little need to dispute the right-of-way with streams. It was obviously safe and conservative to leave the streams in their long-established courses and build around them or bridge over them when conditions became critical. In those years the use of 20- to 56-degree curves was common and was considered acceptable practice in mountain road location.

With the advent of faster traffic, the former alinement standards quickly became obsolete. The traveling public demanded roads capable of serving more safely faster moving vehicles. With the further improvement of motor vehicles, this demand became more and more insistent. Obviously, one important factor in the solution of the problem was the reduction of curvature to a practical minimum.

Many of our primary forest highways are in mountainous country where the most economical location, and often the only feasible one, follows the course of some tortuous stream. Characteristically, such streams alternately flow through narrow, winding mountain valleys and through sharply defined canyons. The general problem of location in such situations is simple since the stream is the major control. The solution involves establishing a proper grade line, fitting the alinement within the limitations imposed by the high-water elevation and the topography of the valley, and determining to what extent, if any, crowding or diverting the stream or crossing it is justified in order to obtain satisfactory alinement.

In deciding whether to introduce channel changes or to leave the stream in its natural bed and use bridges, landscape values must not be overlooked, especially within the national parks or national forests. Bridges of harmonious design with adequate waterways, in general, do little violence to natural topography, but all such structures entail special and perpetual maintenance costs. Of even greater importance, however, is the high first cost of bridges. In contrast, channel changes generally cost less to construct than bridges. The saving is effected chiefly by the use of modern methods of machine excavation. In spite of introducing some additional scar into the landscape, economy and better alinement are more often possible with channel changes; therefore, highway engineers of the Northwest have carefully studied the advantages and disadvantages of using channel changes instead of bridges in locating certain roads.

Channel changes require careful study, especially if a sizable stream is involved. The streams have followed the line of least resistance in eroding their present channels, and equilibrium resulting from all the factors of friction in the channel has been established. Where this equilibrium is disturbed by constructing a steeper and shorter channel, provision must be made for the increased erosive capacity of the stream.

Since the carrying power of moving water varies as the sixth power of its velocity, it is obvious that any change resulting in an increase in velocity caused by shortening and straightening a stream cannot be undertaken in a haphazard manner. The energy of the water must be dissipated in such a way as to prevent destructive erosion. The transportation of channel debris to downstream points where it might be deposited and build up the stream bed enough to flood the roadway and adjacent property must be prevented. The problem is one of duplicating, so far as possible, the friction head in the original channel, or of providing a channel capable of resisting the greater erosive force if a higher current velocity is to be permitted.

SEVERAL PRINCIPLES INVOLVED IN DESIGN OF CHANNEL CHANGES

As applied to forest highway construction in Washington, Oregon, and Montana, the ordinary principles involved in channel change construction are briefly summarized as follows:

1. The highway is located well above maximum high water, allowing a margin of safety to cover factors difficult to evaluate.

2. Material excavated from the channel is used in constructing the roadway.

3. The stream side of embankments and the channel slopes are protected by a 4- to 6-foot layer of angular rock obtained either from the roadway excavation or borrowed and placed as loose riprap. This rock protection is usually placed outside of the finished roadbed prism and thus widens the shoulder on the stream side. The additional width, if considered necessary, may be utilized to support a guardrail, although the extra width in itself is a margin of safety. Seventy-five percent of the riprap material ranges from one-half cubic foot to 1 cubic yard in size. The largest rocks are placed at the bottom and are moved roughly into place with crowbars.

4. Unless ample room is available to effect a wide separation between the channel change and the roadway prism, the roadway embankment slopes are usually designed as part of the channel slopes so that the slope is continuous from the road shoulder to the stream bed. Berms are seldom used between the roadway and the channel because erosion is apt to occur along the berm unless it is carefully protected. The heavy course of loose riprap provides sufficient material so that any undercuts at the toe of the channel slope that may be eroded by the stream are immediately filled with the coarse material, thus effectively preventing further erosion.

5. The channel designed has sufficient width and depth to provide adequate carrying capacity. The bottom of the channel is made sufficiently rough to duplicate the original friction head or is of resistant material that will permit the higher velocity without erosion. Data are taken on the original channel above and below the proposed change, and the new channel is fitted into the old smoothly.

6. If conditions permit, the bank of the new channel opposite the road is cleared and grubbed for a width of 20 to 50 feet to weaken the bank so that if the channel provided proves inadequate the stream will erode that bank rather than the roadway embankment. In areas containing large timber this practice insures that erosion will not undermine the standing timber, causing log jams and consequent destructive erosion.

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7. After construction, maintenance is carefully supervised to see that any deficiencies that develop are corrected. In some instances increased friction is provided by placing impediments such as large, angular rock in the channel bed. In other cases the channel is widened to reduce the velocity and also increase the friction.

It must be borne in mind that stream flow is a powerful force very difficult to evaluate. It would be surprising if some failures were not caused by the extreme floods that occur once in 20 to 50 years. It has been the history of railroads and highways that such floods take toll of bridge structures as well. It is, therefore, to be expected during critical floods that some small failures of channel revisions will be experienced. Failures of channel changes are most likely to be minor in character and can be repaired at small expense. After repair the weak features will have been eliminated and the road should be safe for many years.

On roads of high standards in difficult terrain it is sometimes cheaper to construct channel changes than to build bridges, and, if large savings in first cost are possible, assumption of the additional risk would seem to be warranted. Even though they may be damaged in some degree by occasional, unusual floods, it is often sounder economic policy to build highways at moderate cost, making use of channel changes where reasonably safe, than to build expensive, ultra-conservative roads and bridges that will withstand all floods. Since obsolescence is an important factor influencing the useful lives of highways and future traffic demands are often difficult to foresee, it will usually result in ultimate economy to design roads and bridges to withstand floods normally to be anticipated and rebuild them after damage by infrequent, abnormal floods. The large mileage of roads needing improvement and the limited funds available favor this policy.

In any locality, the inclusion of channel changes in highway design will depend on a thorough understanding of the characteristics of the streams involved and of the tributary watershed. While channel changes have proved their economic worth on many projects in the Northwest where small or moderate-sized streams are involved, and where the regimen of the stream is not too severe, it is recognized that there are many locations where channel changes would be distinctly hazardous. Channel changes cannot be used indiscriminately.

Preservation of landscape values should be a cardinal principle in any highway design. It is recognized so far as appearance is concerned that seldom can man-made water courses improve on nature. Careful planning, however, can minimize the artificiality of channel changes. Vegetation, encouraged by ample rainfall, will quickly cover the more noticeable construction scars, but some of the attractive characteristics of a natural stream unavoidably will be lost. It is obvious, therefore, that if an alternate location exists comparable in standards and costs which does not involve channel changes, the alternate is to be preferred.

EXAMPLES SHOW ECONOMY OF BUILDING CHANNEL CHANGES INSTEAD OF BRIDGES

The following examples (projects A and B), for which alternate design data are available, are cited to give interesting comparisons. Both are on important routes where high standards of alinement were considered justified. Design data are included for a third example (project C) involving major channel changes, but no comparable alternate location was possible due to the terrain.

Project A.—This project is on both the Federal-aid and Forest Highway Systems in Oregon on one of the more important transmountain highways. It is the shortest and easiest route from the Willamette Valley and points north to California and probably will divert a portion of the traffic now carried by other parallel routes. It is estimated that this highway will carry from 500 to 1,000 vehicles per day.

The original design for the section discussed here involved curves of 10 degrees or less without channel changes. The alinement on each side for some distance consisted of long tangents and long-radius curves. A period of several years intervened between the original location survey and construction. When the section was finally proposed for construction alternate designs were studied with the result that, although the cost was increased by approximately $11,000 per mile, the benefits derived by using channel changes were considered to justify the additional expense on the 1.8 miles affected. The original design involving the sharper curvature was considered dangerous and inadequate.

Table 1 gives a comparison of the two lines. A rough estimate showed that a design using bridges and minor channel changes would have cost an additional $28,000 per mile.

Five channel changes on project A will be illustrated and described briefly.

TABLE 1.—*Comparison of curvature data for project A (1.8 miles long) as originally surveyed and as finally constructed*

Line	Total curves	Total angle[1]	Maximum curve	Number of curves of—						
				2°	3°	4°	7°	8°	9°	10°
	Number	Degrees	Degrees	Number	Number	Number	Number	Number	Number	Number
Original [2]	9	378	10	1	---	1	-----	2	2	3
Constructed [3]	6	194	7	1	2	1	2	--	-----	-----

[1] All curves on the constructed line are spiraled, and spiral angles are included in the total angle for the constructed line.
[2] Without channel changes.
[3] With channel changes.

Figure 1, A shows the original terrain, looking upstream from station 1725, and figure 1, B shows the completed channel change and road. The road has a curve of 6°20′ and a grade of 5.5 percent. The grade of the road exceeds the grade of the stream bed so that the road gradually rises above the stream.

The channel is 38 feet wide at the bottom, and its slopes are 1 to 1. The highway fill slopes are 1½ to 1. Sufficient angular rock for slope protection was obtained from the adjacent excavation without expense other than the cost of the unclassified excavation involved. The new channel required approximately 5,000 cubic yards of excavation and is 600 feet long.

Figure 2, A shows the original terrain looking downstream from station 1728, and figure 2, B shows the completed channel change and road. The road has a curve of 6°30′ and its grade increases from 3.5 to 5.2 percent. The grade of the road exceeds the grade of the stream bed so that the road gradually rises above the stream.

The channel is 38 feet wide at the bottom. The new channel is 500 feet long and required 2,100 cubic yards of excavation. Rock for slope protection was obtained

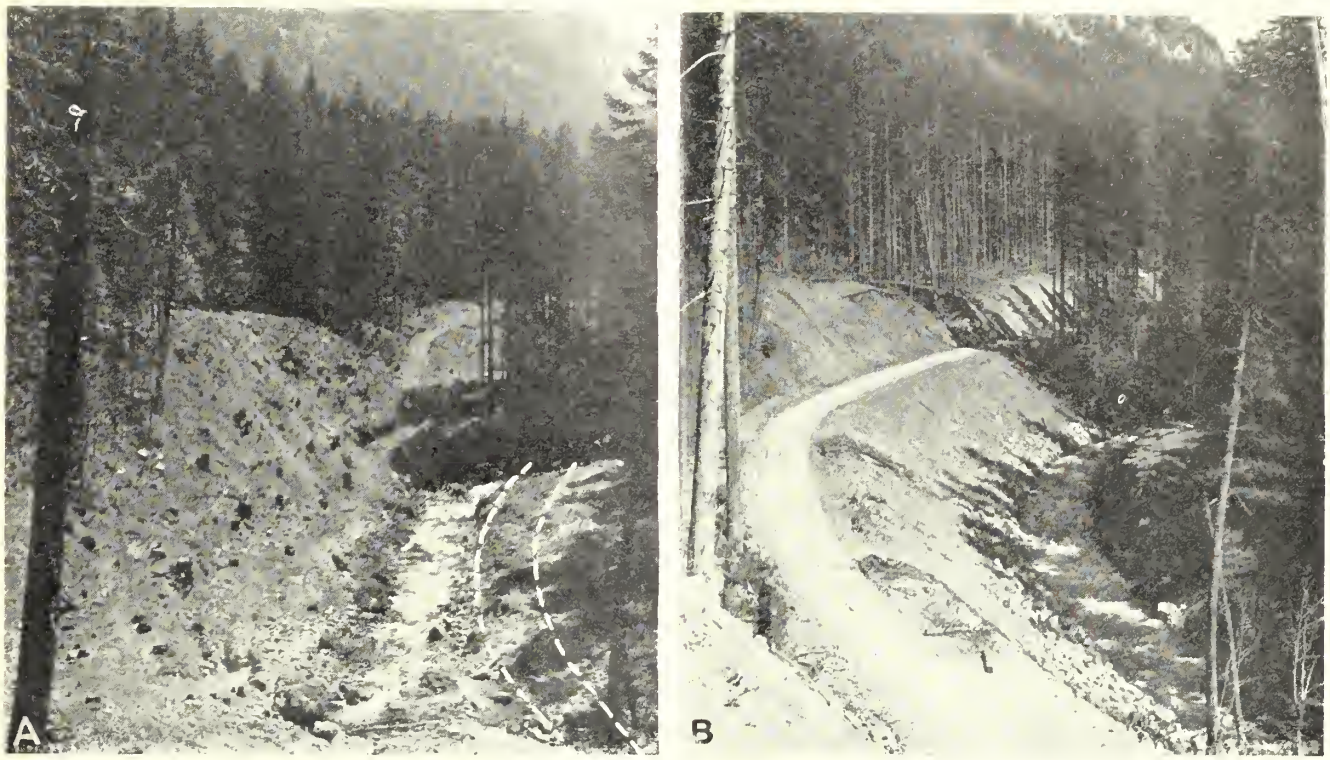


FIGURE 1.—ORIGINAL TERRAIN AND COMPLETED ROAD AND CHANNEL CHANGE. A, THE DOTTED LINES MARK THE CHANNEL CHANGE LOCATION. B, THE HIGHWAY FILL SLOPE IS RIPRAPPED AT THE BOTTOM WHERE IT SERVES AS ONE BANK OF THE STREAM.

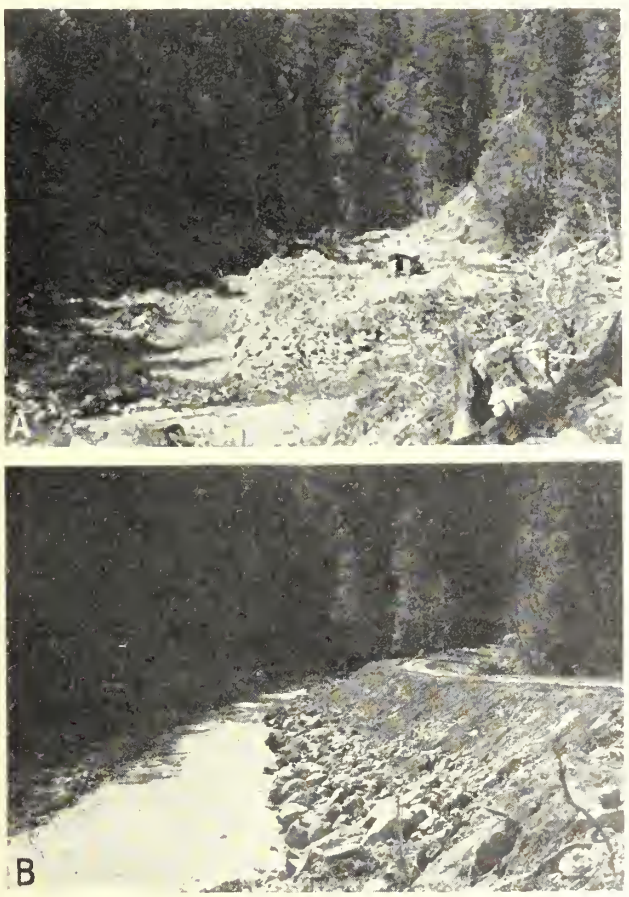


FIGURE 2.—A, ORIGINAL TERRAIN; AND B, COMPLETED CHANNEL CHANGE.

FIGURE 3.—A COMPLETED CHANNEL CHANGE THAT DIVERTS THE STREAM FROM ITS ORIGINAL CHANNEL ENTIRELY.

from adjacent roadway excavation. The stream was diverted entirely from its original channel.

Figure 3 shows a completed channel change looking downstream from station 1738. This channel change, which diverted the stream from its original channel entirely, is 1,100 feet long and involved 22,000 cubic yards of excavation. The channel change cost approximately $9,000 and made possible an alinement that otherwise would only have been possible by constructing two bridges at a probable cost of $25,000.

Figure 4, A shows the original terrain, looking downstream from station 1766, and figure 4, C shows the completed channel change and road. Figure 4, B shows operations during construction of the new channel.

This channel change is 600 feet long and required about 6,000 cubic yards of excavation. Most of the rock for slope protection was obtained from adjacent

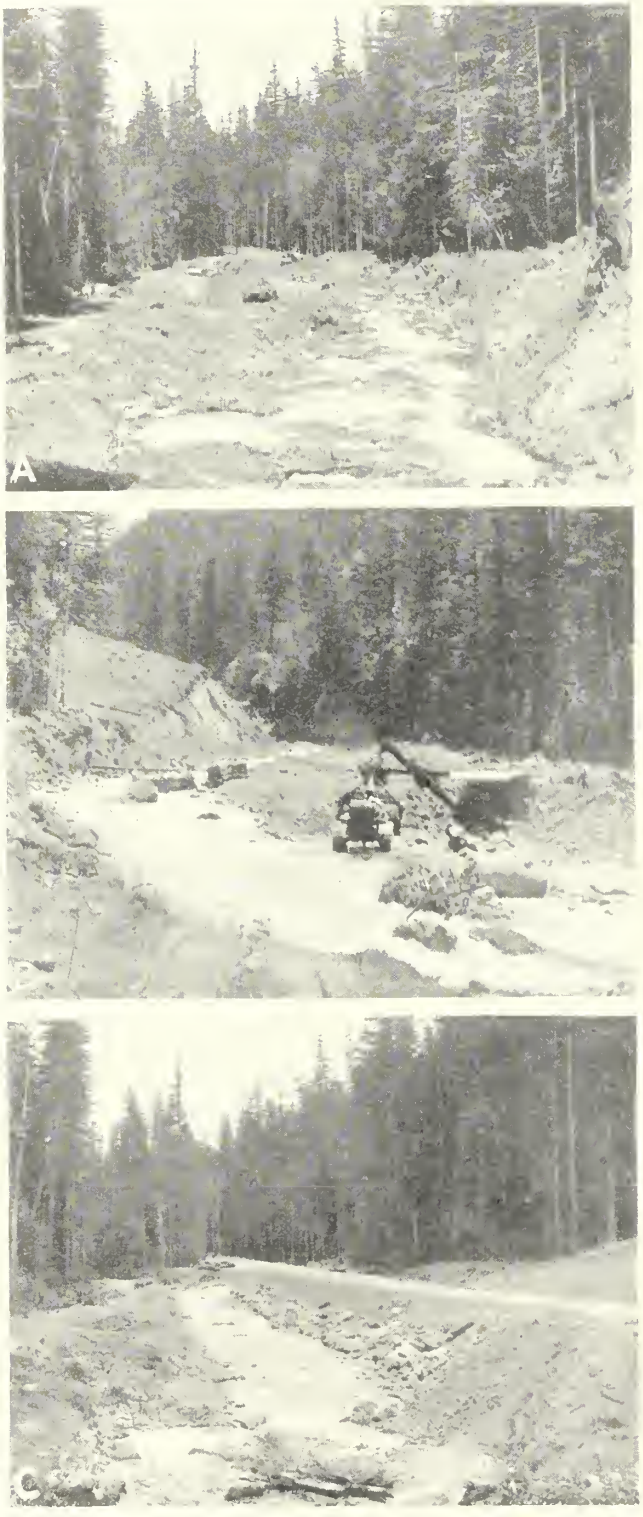


FIGURE 4.—A CHANNEL CHANGE AT VARIOUS STAGES OF CONSTRUCTION: A, ORIGINAL TERRAIN AS CLEARED OF TREES AND UNDERGROWTH; B, CONSTRUCTION OPERATIONS; AND C, THE NEW CHANNEL COMPLETED.

cuts. The left bank of the stream (fig. 4, C) has been cleared and grubbed for a width of about 50 feet.

The completed channel change shown in figure 5, A is 4,000 feet long. This picture shows the channel change looking upstream from station 1943. About 50,000 cubic yards of roadway embankment were required, 30,000 cubic yards of which were obtained from the channel excavation, and 20,000 cubic yards from cuts at each end.

About 3,000 cubic yards of loose riprap were placed in addition to rock brought from adjacent cuts. The riprap (fig. 5, B) is a conglomerate rock of fair quality, ranging in size from one-half cubic foot to one-half cubic yard. This rock was dumped by trucks and moved into place by crowbars.

Note how the channel has widened and material has been deposited at the lower end (foreground, fig. 5, A). This was caused by flattening of the stream grade and by drift. It is expected that the next floodwaters will scour the channel enough to remove this deposit.

CHANNEL CHANGES OFTEN ECONOMICAL MEANS OF ATTAINING HIGH STANDARDS OF ALINEMENT

Project B.—This project is on a route that crosses the Cascade Range. It follows practically a water grade from the Willamette Valley to a point 3 miles from the summit of the Cascade Range, which it climbs on a 5.5-percent grade, and then descends for 5 miles on a 5-percent grade to the central Oregon Plateau. This route affords one of the most favorable crossings of the Cascades and is expected to carry considerable traffic.

Six years ago a survey was made and a design prepared that involved no major channel changes or bridges. This design required several curves ranging from 14° to 22° and represented the best alinement possible without using bridges or major channel changes. In later surveys, consideration of possible stream crossings or channel changes revealed that much better alinement was possible. The best obtainable alinement was obtained by making major channel changes, and cost less than the design involving bridges.

No direct cost comparison with the original design is available since the surfaced road widths in the designs were different. It is estimated that the road as constructed cost approximately 50 percent more than if it had been constructed according to the original design for the 5.6 miles where the stream was the control. To improve the original alinement appreciably without bridges or channel changes was impracticable. The alinement of the highway on both sides of the section for several miles is very good and to have constructed a road with the alinement first considered would have resulted in a bottle neck that would retard traffic. Table 2 compares the alinement for the three designs.

The line as originally surveyed followed the stream closely, thus reducing construction costs but necessitating sharp curves. The maximum curvature on the revised line was 6°, but this line required four bridges and several channel changes. Additional channel changes were made to eliminate the bridges on the constructed line.

Figure 6 shows a channel change on project B during construction and after completion. Figure 7 shows several completed channel changes. Figure 8, A shows a channel change and highway fill nearly completed, and figure 8, B shows a completed channel change and highway fill protected by riprap.

Project C.—This project is on the road between Canyon City and Bear Gulch, Oreg., and follows the floor of Canyon Creek for a distance of 8 miles. In many places there is scarcely room for both the stream and the highway. It was impracticable to work out any acceptable alinement without channel changes. Even with extensive channel changes the best alinement that could be obtained involved 26 curves ranging from 10° to 28°, with 6 curves exceeding 20°.

Material excavated from the channel change was used to construct the road.

Coarse rock helps prevent erosion of the highway fill.

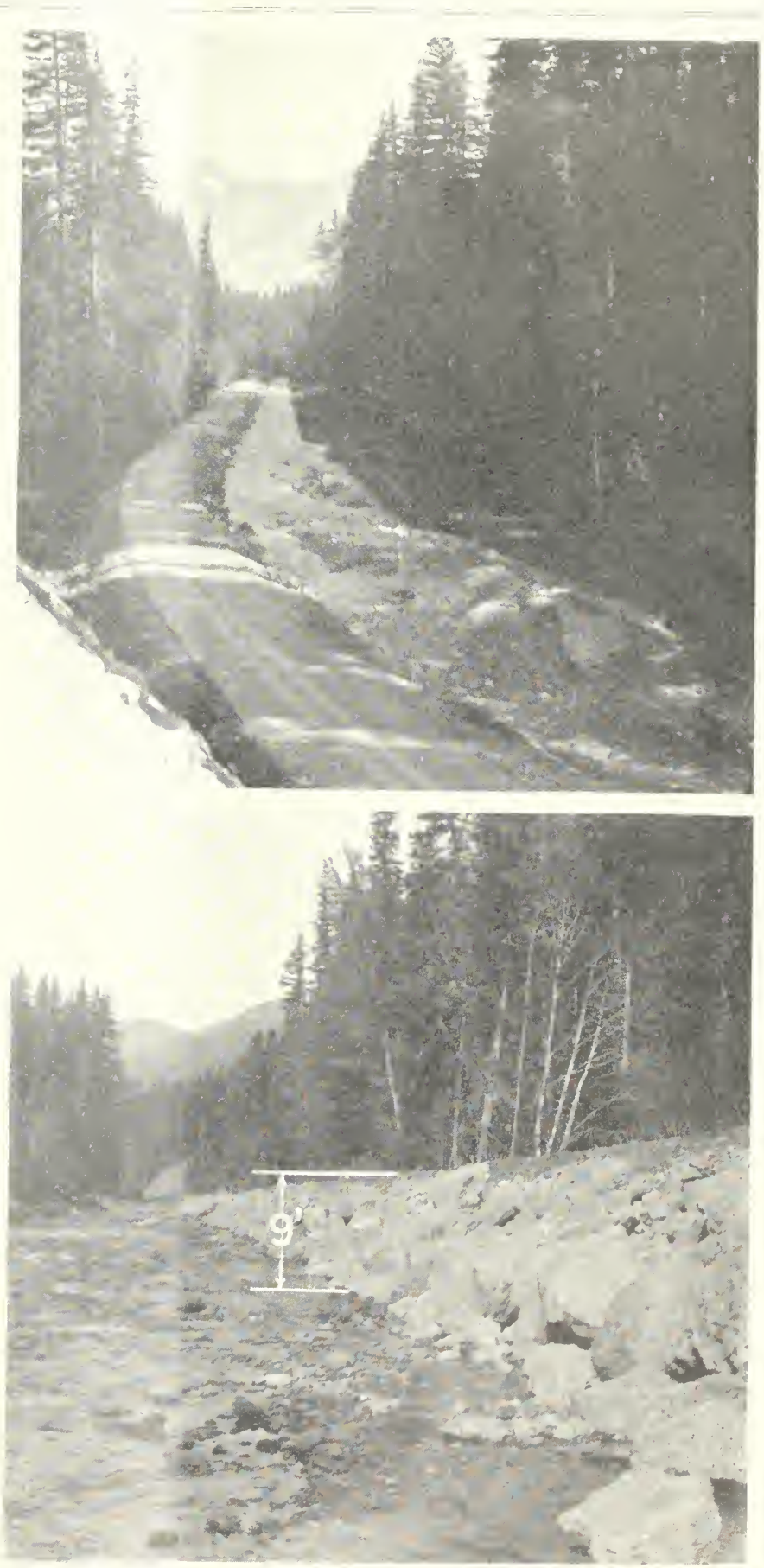
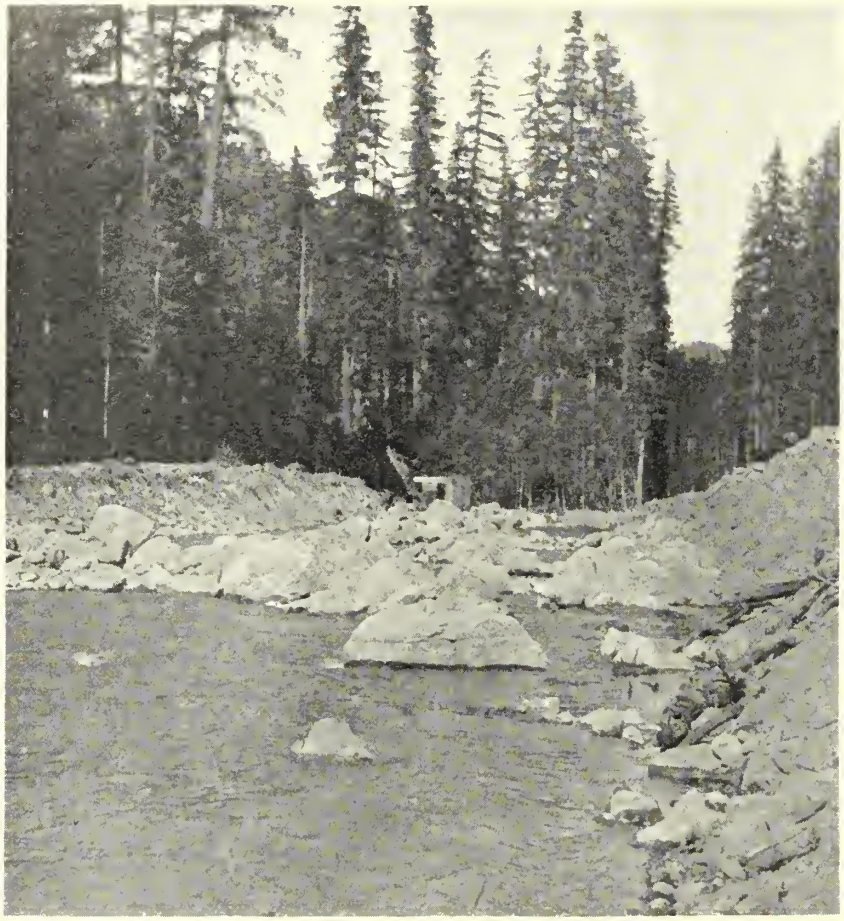


FIGURE 5.—TWO VIEWS OF A COMPLETED CHANNEL CHANGE.



The large boulders remaining in the stream were later used for slope protection.

Lower, the left bank of the channel is cleared of vegetation to permit erosion.

FIGURE 6.—PROGRESS DURING CONSTRUCTION OF A CHANNEL CHANGE.

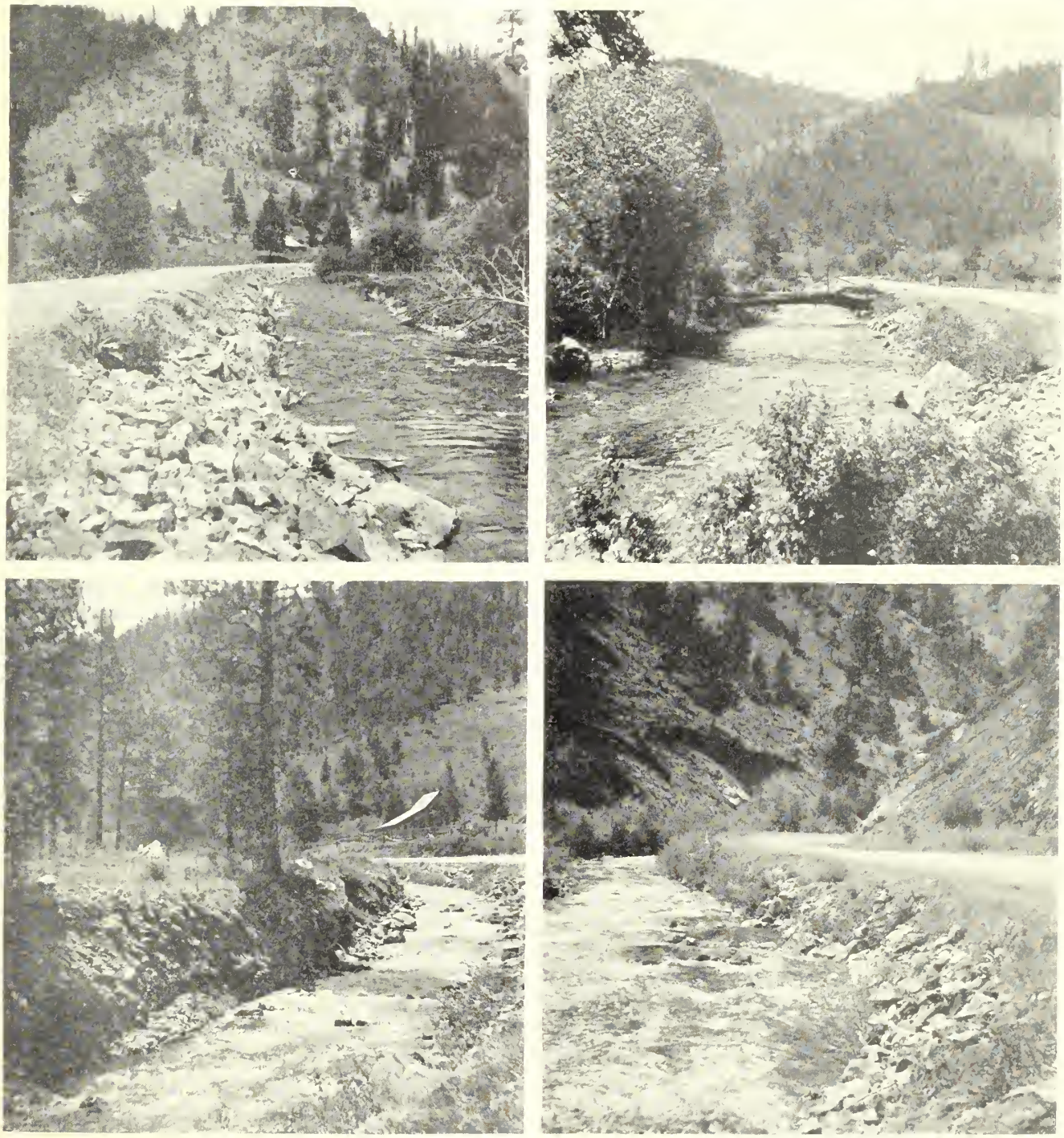


FIGURE 7.—TYPICAL CHANNEL CHANGES USED TO BENEFIT ROAD LOCATIONS.

The grading of the road was completed in 1931 and it has now gone through 6 years of winter and spring floods without signs of damage. However, there have been no unusually high floods.

No data are available as to comparative costs of alternate designs. The material from the channel excavation was used in building the roadbed. It was largely gravel and boulders and was handled with power shovels and trucks at a contract price of approximately 40 cents per cubic yard. The adjacent hillside material is largely rock which at the time the work was done would probably have cost approximately 75 cents per cubic yard to move. Quantities on the adopted design were much lighter than could have been obtained by sidehill construction. It is obvious that the cost of the road as built was considerably less than would have been the case with sidehill construction and bridges, and the alinement is better. Figure 7 shows several channel changes on this project and the loose rock protection used on the highway fill slopes extending into the stream.

(Continued on page 182)

EXPERIMENTAL EROSION CONTROL ON FOREST HIGHWAY FILLS

BY DISTRICT 2, BUREAU OF PUBLIC ROADS

INTEREST in the development of practical methods of erosion control has grown tremendously during the past several years. Until recently a relatively unexplored field as related to highways, engineers are now experimenting with various methods of preventing erosion on highway fills.

Experimental work on forest highway fills in California, though somewhat limited in extent and so recent that conclusive evidence of the effectiveness of the various types of control is not yet available, nevertheless gives some indication of the results to be expected. The purpose of this work has been to determine the most practical methods of preventing erosion and encouraging revegetation on newly constructed fill slopes under varying soil and climatic conditions. Types of treatment have varied, from broadcasting grain and other seeds on the slopes or covering with forest duff, to more extensive methods using various types of revetments or wattling.

The landscaping of roadsides to eliminate unsightly construction scars and to give the roads a more pleasing appearance has, within the past decade, been stimulated in all States by a provision of highway legislation enabling Federal funds to be spent for such purposes. This landscaping has included the planting of desirable vegetation along roadsides. The plantings have served to help prevent erosion, though in this work the erosion control effect was generally subordinated to the roadside beautification objective.

Work of this character was performed on a project in the Tahoe National Forest, in Sierra County, Calif. Crested wheat seed was sown broadcast on two fills late in the fall of 1935. Figure 1 shows one of the fills 8 months after planting. The soil in the fills was not very erosible, and the chief value of the planting was to screen the bare earth and thereby improve the appearance of the roadside.

More extensive methods of erosion control were used on three other forest highways in California as follows: On route 20, in the Plumas National Forest in Plumas County; route 61, in the Angeles National Forest in Los Angeles County; and route 74, in the Sierra National Forest in Madera County. Conditions on these three projects represent almost the extremes where extensive erosion control methods appear feasible or desirable.

Conditions on route 20 may be considered to represent minimum needs. Clay predominates in the soil structure, though there is some rock that produces a certain amount of stability. Although the precipitation is heavy, it falls as snow which melts gradually and does not cause concentrated run-off.

Conditions on route 61 represent the other extreme. Here the soil is extremely erosible and most of the precipitation falls as rain in storms that are often cloudbursts. Furthermore, run-off from the highway enters streams that furnish water for irrigation and city water supplies, and it is imperative to avoid filling them with debris.

The methods of erosion control used on these three projects and the results obtained will be discussed in some detail.

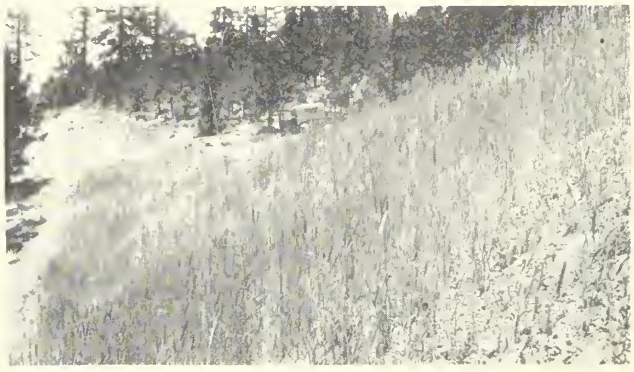


FIGURE 1.—A HIGHWAY FILL 8 MONTHS AFTER CRESTED WHEAT HAD BEEN PLANTED.

WILLOW CUTTINGS PLANTED ON ROUTE 20

Route 20 is located in northeastern California, and immediately following the completion of the road in 1935 the erosion control work was started.

Climatic conditions in this locality are not ordinarily conducive to excessive erosion, as the snow melts slowly since warm rains are infrequent. The soil is chiefly clay, with some rock. From 35 to 55 percent of the soil passes a 200-mesh sieve. The large amount of fine material in the soil indicated that saturation of the newly constructed embankments might possibly result in the loss of considerable material by mud flows or slides. Surface scour was not considered an important factor because of the absence of heavy rainfall and the rapidity of reproduction of native vegetation.

In view of the favorable conditions existing on this road, the erosion control considered necessary involved stabilizing the fill slopes to an adequate depth below the surface. This was done by planting willow cuttings 3 feet long to a depth of 30 inches in the fill slopes. The cuttings were planted in rows following contour lines, spaced at vertical intervals of about 3 feet between rows. The spacing of the cuttings in rows varied from 1½ feet on the highest fills to 3 feet on small fills.

The appearance of one of the fills after planting had been completed is shown in figure 2.

For experimental purposes, on large fills the cuttings were supplemented by brush wattles placed in trenches directly above the cuttings. These trenches were 1 foot wide and 1 foot deep, and were filled with willow limbs and brush having an average length of 3½ feet and maximum butt diameter of 2 inches. The limbs were matted together to form a continuous chain along the trench, and were covered with a layer of small, newly cut fir limbs. Sufficient earth was then shoveled on top to hold all material in place. Further compaction was obtained by workmen walking on the wattles during construction operations. A fringe of the newly cut fir limbs was allowed to protrude above the surface in order to disperse surface run-off.

Approximately 0.4 slope acres of the total of 4.1 acres treated were supplemented with brush wattles. The fill slopes were then seeded with wheat and native dock.

The costs of the work for the various spacings of cuttings were as follows:

Spacing, feet	*Cost per slope-a.*
1½	$332
2	250
3	166

Willow cuttings spaced 1½ feet apart and supplemented with brush wattles cost $847 per slope acre.

In spite of the precautions taken some minor slips occurred. Figure 3, A shows minor slips on a fill after the winter season of 1935–36. Figure 3, B shows the same fill after having been repaired.

Excellent results were obtained on most of the fills planted with willow cuttings. Figure 4 shows a fill that had no failures. Although some of the plants shown in this figure may die, the majority should flourish and afford ample protection for revegetation by indigenous shrubs and small plants.

FIGURE 2.—WILLOW CUTTINGS PLANTED ON A HIGHWAY FILL TO PREVENT EROSION.

BRUSH LAYERS PLACED IN FILL SLOPES DURING CONSTRUCTION

Route 61 is located in southern California. Soil and climatic conditions are such that extensive erosion has occurred on the fill slopes of nearby forest highways. Heavy rains and snowfalls, the latter melting quickly under warm rains and temperature, result in heavy and rapid run-off. Extensive erosion occurs even on the steep mountainsides despite native cover. Winds of high velocity also cause erosion in this locality.

It was thought advisable to perform the erosion control work during construction, since the unusually high fills proposed would expose large areas of loose material to the heavy rains common to the region. The work was performed during the winter and spring of 1935–36.

The soil with which the fills were constructed consisted of disintegrated granite and disintegrated schist, with some harder materials. It was found that the soils could be classified according to composition as follows:

Class 1.—Disintegrated schist or granite; 95 percent passing the 2-inch sieve; 35 to 56 percent passing the 200-mesh sieve.

Class 2.—Disintegrated schist or granite and rock; 50 percent passing the 2-inch sieve; 10 to 25 percent passing the 200-mesh sieve.

Class 3.—Rock; 25 percent passing the 2-inch sieve.

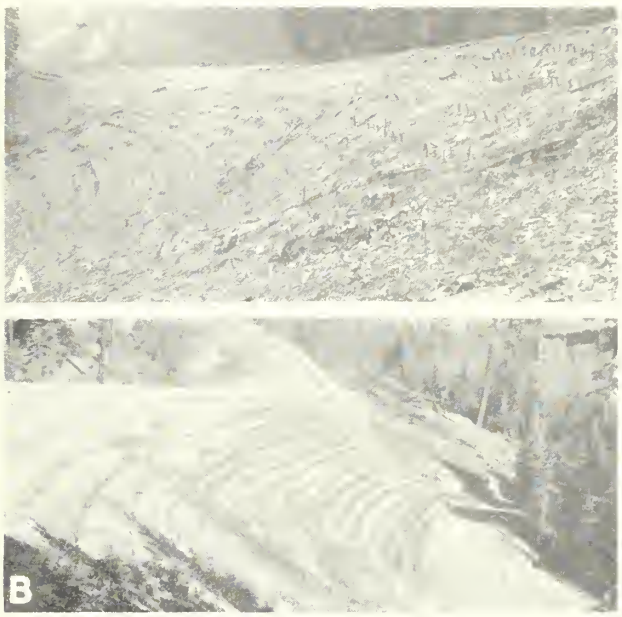


FIGURE 3.—A; MINOR SLIPS OF WATTLES AND CUTTINGS THAT OCCURRED ON A HIGHWAY FILL. B; THE FILL SHOWN ABOVE AFTER HAVING BEEN REPAIRED.

Fills composed of class 3 material were not treated. Three kinds of protection work were used on this project.

Method A.—Brush in fill layers.—All suitable brush obtained from clearing operations was stockpiled for future use in erosion-control work. This brush was placed, during construction of the fill, along the outer edge of the compacted layer in rows along contour lines. The distance between rows depended upon the height of the fill. In general, a 5-foot spacing was used on that portion of a fill lying less than 40 feet below grade; a 4-foot spacing was used from 40 to 70 feet below grade; and a 3-foot interval was used on all portions of fills lying 70 or more feet below grade.

FIGURE 4.—APPEARANCE OF A FILL 9 MONTHS AFTER WILLOW CUTTINGS HAD BEEN PLANTED.

Each fill was compacted in layers with the outside edge slightly higher than the center. Brush was placed while the center of the fill was being compacted, thus avoiding interference with grading operations.

Long-stemmed brush or small logs were first placed along the edge of the layer, parallel to the centerline of the road. Brush was then placed with stems inward and at an angle of about 45 degrees with the edge of the fill. The branches protruded from 12 to 24 inches beyond the fill slope, and the stems extended into the

fill from 2 to 6 feet, depending on the length of brush available. Small brush, roots, and small stumps were used to fill in and complete the mat. After the brush layer had been placed, a bulldozer was used to cover the brush out to the slope line.

The placing and covering of brush layers are shown in figure 5.

Particular attention was paid to extending the stems well into the fill to insure stability and prevent slipping or flowing of saturated surface material.

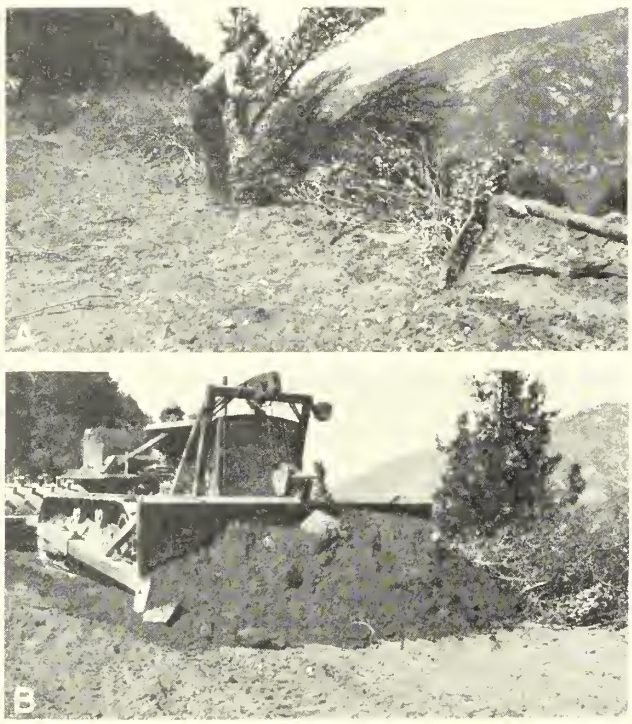


FIGURE 5.—A; PLACING A LAYER OF BRUSH. B; A BULLDOZER COVERING A BRUSH LAYER WITH EARTH. THE BULLDOZER IS PULLING A SHEEPSFOOT ROLLER USED TO COMPACT THE FILL.

Method B.—Brush in fill layers with hay mat.—Brush layers were first placed during construction of the fill, according to method A. The area between brush rows was then raked and smoothed, and alfalfa, barley, or oat hay was spread, beginning at the top of the fill and working downward by rows. After placing the hay, common rye seed was sown and a thin layer of earth was shoveled onto the hay from the area just below the brush row. The earth cover was intended to hold the hay in place, cover the seed, and discourage feeding by deer. After placing the earth cover, Italian rye and Australian rye seed were broadcast over the entire area of the fill slope.

Hay was spread at the rate of 6 tons per slope acre; common rye seed was sown at the rate of 100 pounds per slope acre; and Italian or Australian rye seed at the rate of 40 pounds per slope aere.

Method C—Stake and brush wattles with hay mat.—At locations where brush was not readily available during construction, fill slopes were treated after completion.

Fill slopes were first smoothed, and rows of 2-inch by 2-inch by 42-inch stakes were driven into the fill to a depth of 34 inches and normal to the slope. The rows were placed on contour lines at intervals of 3 to 5 feet, and individual stakes were placed 3 feet 3 inches apart in the rows. A level path approximately 1 foot wide was then excavated immediately above each row and brush was placed horizontally above the stakes to form a wattle.

The brush consisted almost entirely of manzanita, buck brush, mountain lilae, scrub oak, and greasewood. An effort was made to select brush with reasonably straight stems and a number of small, leafy branches. The length varied from 3 to 4 feet, and butt diameter from 1 to 2 inches. For some fills, suitable brush could be cut in the immediate vicinity; for others, it was necessary to haul it in by truck.

The brush was interlaced and compacted to form a wattle 1 foot wide and 1 to 1½ feet high. The entire thickness or height extended above the surface of the fill, and the wattle rested against the supporting stakes. After completion of the wattles the area between rows was covered with hay, seeded, and covered with earth as described under method B.

Table 1 shows the areas treated by each method and the costs.

Additional protection work not shown in table 1 was done in the summer and fall of 1936.

Method A was used in treating fills of class 2 soils, and methods B and C were used in treating fills of class 1 soils.

TABLE 1.—*Areas of fill slopes treated by three methods of erosion control, and cost per acre for each method*

Method	Area of slope treated	Cost per acre
	Acres	
A	2.7	$213
B	10.5	449
C	3.7	715

FAILURE OF BRUSH WATTLES CAUSED BY SLIPPAGE OF SURFACE MATERIAL

Some of the protection work was damaged by herds of deer. Probably because of the scarcity of forage in the surrounding mountains, the deer fed on the hay placed on the fills. In some locations this resulted in the complete loss of the hay mat and considerable damage to the rest of the protection work. It was deemed advisable to replace the hay and reseed the slopes prior to winter storms, so this was done. Also, approximately 300 Yerba Santa root euttings were planted per slope-acre on slopes with southern exposures. This plant is native to the locality. Its growth was particularly vigorous under conditions similar to those existing on the fills where it was planted. All slopes were seeded with Italian rye and burr clover, with some acorns on slopes with northern exposures.

The protected fills were subjected to a severe test in February 1936 during construction. Approximately 10 inches of rain fell, at times approaching the rate of 1 inch per hour. After this storm, damage to protected and unprotected slopes was noted. (See figs. 6, 7, and 8.)

Fills protected by brush rows and hay mats eroded the least. Brush rows without hay mats were less effective protection, although they tended to break up small mud flows and prevent the formation of large channels.

Results to the middle of the winter season 1936–37, following unusually severe winter conditions, show considerable erosion of untreated fill slopes. There have

FIGURE 6.—APPEARANCES OF PROTECTED AND UNPROTECTED FILL SLOPES AFTER A HEAVY RAIN.

been some slips of wattled fills, but little erosion or loss of material has occurred on brush-treated fills. Figure 9 shows damage to fills protected according to method C. No protection work was performed on the upper sections of the fills shown in figure 9 because of the rocky material present.

The failure of wattles on some fills was probably caused chiefly by slippage of the surface material. Apparently the wattles were not as well anchored to the fill as were the rows of brush laid according to methods A and B.

The results obtained by the protection work, as determined by an inspection made in March 1937, are shown in table 2.

TABLE 2.—*Results obtained on fill protection work on route 61*

Method	Treated area inspected	Area of failures	Percentage of failure
	Acres	*Acres*	
A	1.7	0.4	24
B	11.5	1.2	10
C	4.8	2.7	57

FIGURE 7.—FILL SLOPES PROTECTED ACCOPDING TO METHOD A. PICTURE SHOWS A SECTION ABOUT 70 FEET BELOW GRADE.

EXTREMELY EROSIBLE SOIL IN FILLS ON ROUTE 74

Route 74, located in east-central California, was constructed during 1933 and 1934.

The soil on the fill slopes was not favorable to plant growth and this, together with erosion, prevented revegetation. The soil consisted of disintegrated granite with various percentages of hard rock particles. The soil was also micaceous in various degrees, the

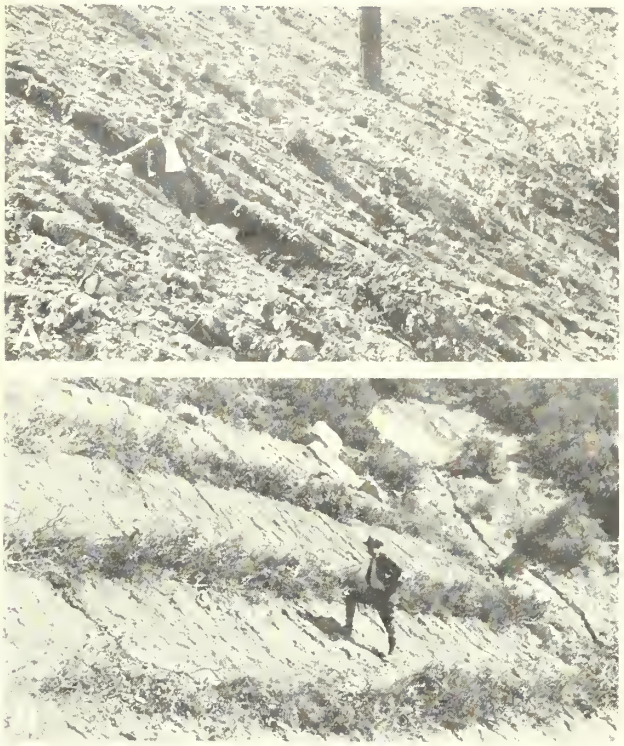


FIGURE 8.—SECTIONS OF FILL SLOPES ABOUT 70 FEET BELOW GRADE. A; EROSION ON AN UNPROTECTED FILL. B; A FILL PROTECTED ACCORDING TO METHOD A.

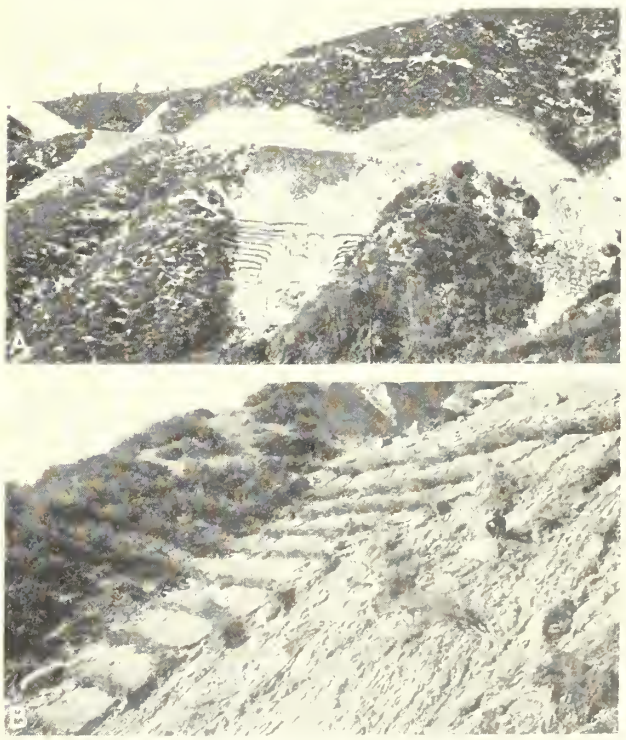


FIGURE 9.—DAMAGED FILLS THAT HAD BEEN PROTECTED BY BRUSH WATTLES (METHOD C).

mica content ranging from a trace to a maximum of 20 percent, and was extremely erosible. The grading of the soil was approximately as follows:

	Percent
Passing the 2-inch sieve	75–100
Passing the 200-mesh sieve	20–40

A typical example of gulley erosion in this type of soil is shown in figure 10. Practically all of the higher fills have northern exposures. Because most storms come from the north, erosion is greatest on the fills having northern exposures and least on fills having southern exposures.

An outline of the proposed control work on route 74 was prepared by the Landscape Division of the Forest Service, Region 5, and the work was done under the direct supervision of a Forest Service foreman with previous experience on similar work.

In general the methods of construction used were the types outlined in the May 1936 edition of "Specifications for Erosion Control Methods", Manual of Region 5 of the Forest Service. The types used will be described in detail.

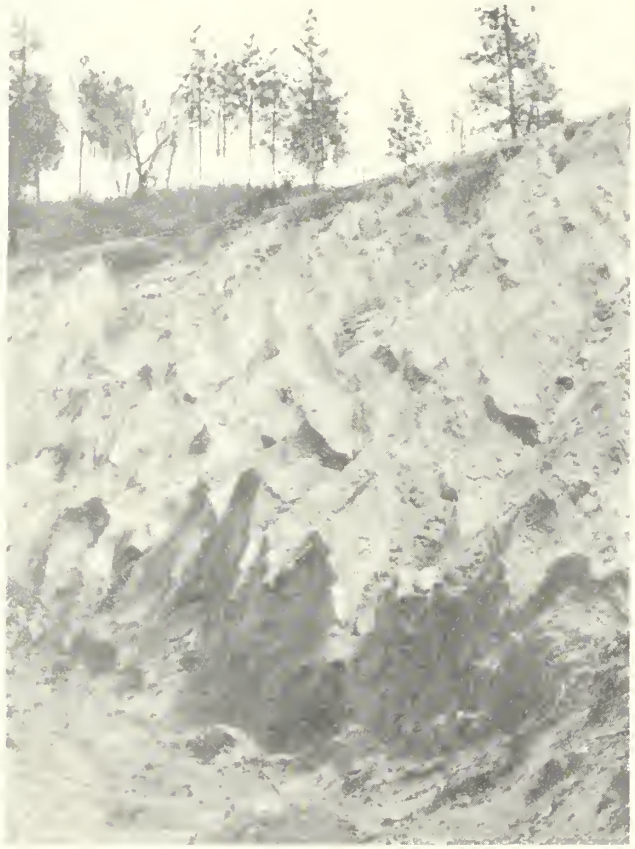


FIGURE 10.—EXAMPLE OF GULLEY EROSION OCCURRING IN SOIL SIMILAR TO THAT ON FILL SLOPES ON ROUTE 74.

Wattling method A (*brush wattles and hay or straw*).—This method was used on the higher, more exposed fills and consisted of brush wattles anchored with 3-foot stakes driven 30 inches into the fill. The brush consisted chiefly of very leafy chaparral and mountain lilac, in lengths of about 4 feet and with a maximum butt diameter of 1 inch. A trench 1 foot wide and about 1 foot deep was dug immediately above the stake row. The brush was laid and compacted into a wattle 1 foot wide and 1½ feet thick. The completed wattle extended 6 inches above the surface of the fill slope. Sufficient earth was placed on the wattles after installation to hold the brush in place.

The spaces between wattles were covered with either hay or straw. Distances between rows of wattles and between stakes in the rows were varied. It was originally intended to space rows on contour lines 3½ feet

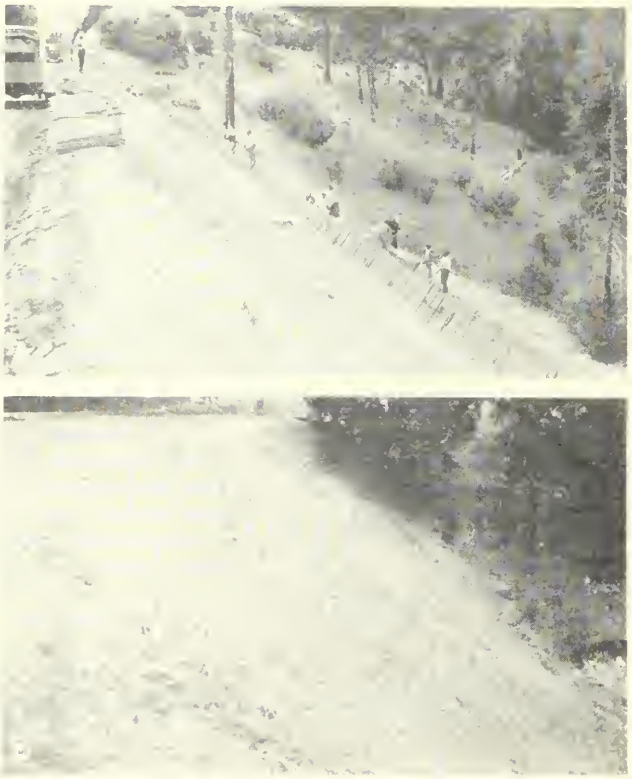


FIGURE 11.—A; STAKES BEING DRIVEN INTO A FILL IN BUILDING BRUSH WATTLES. B; A FILL PROTECTED BY BRUSH WATTLES AND HAY.

apart and to place stakes 18 inches apart in the rows. This spacing was used on approximately 2 acres of the total of 3.6 acres protected by this method. Figure 11 shows stakes being driven in a fill and a fill with protection work completed.

Because construction with the rows 3½ feet apart and stakes 18 inches apart was too rapidly depleting available funds, distances between rows and stakes on the remaining 1.6 acres were increased. The distance between rows was increased to 4 feet (on a few small fills to 4½ and 5 feet) and stakes were driven 2 feet apart in the rows.

Costs per slope acre were $828 for the closer spacing and $620 for the wider spacing.

After being subjected to intense rains during the winter 1936–37, fills on which the closer spacing was used did not appear to be definitely superior to fills with wider spacing of rows. Failures or partial failures by slipping of the wattles amounted to approximately 18 percent on the closer spaced sections, and to 8 percent on the wider spaced sections. However, as the closer spacing was used on the more exposed and larger fills, these percentages cannot be directly compared. Failures of fills protected by brush wattles and hay are shown in figure 12.

Fourteen slopes on route 74 were protected by the brush wattles and hay or straw.

Wattling method B (*hay wattles*).—Hay wattles were placed on nine slopes having a total area of 1.3 slope acres. Six of these fills had southern exposures, and three were on stable-appearing fills having northern exposures. Stakes were driven on contour lines, with a vertical interval of 3 feet between rows; and were spaced 2 feet apart in individual rows. These stakes were 3 feet long and were driven 30 inches into the fill. A shallow trench about 1 foot wide was dug immediately

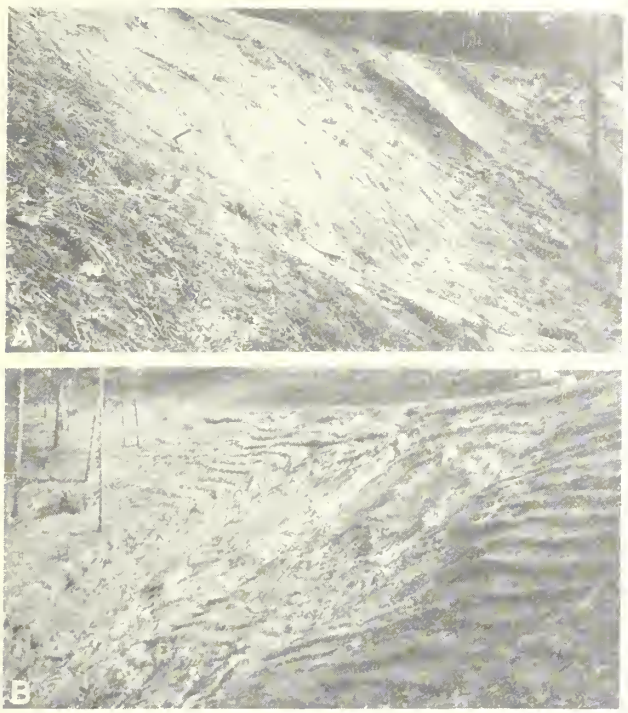


FIGURE 12.—FAILURES OF FILL PROTECTION WORK. THE FILL SHOWN IN B IS THE SAME FILL AS IS SHOWN IN FIGURE 11, B.

above each row and the wattle was placed in this trench. The wattles were formed by twisting and compacting oat hay and straw into a continuous bundle 4 inches in diameter. The wattles were lightly covered with earth to hold them in place. The entire slope was then covered with a layer of straw 4 inches deep.

The cost per slope acre was $256. Fills protected by this method withstood the winter rains remarkably well.

Some slipping occurred on one of the large fills having a northern exposure. This failure, which amounted to 25 percent of the area of the fill, represented less than 4 percent of the total area protected by hay wattles. Partial failure of a fill protected by hay wattles is illustrated in figure 13.

Although fills protected by hay wattles were more favorably situated than were those protected by brush wattles, the former proved satisfactory and might possibly give good results if used on more exposed fills.

Duff method.—In most cases the fills were trenched before duff or litter was placed. The duff consisted of pine needles, dead leaves, small twigs, small pieces of bark, grass, and leaf mold or other humus. This material was scraped from any available areas and spread on the fills to an average depth of 1 inch. As work progressed duff became increasingly difficult and expensive to obtain in sufficient quantity to complete the work.

The scarcity of material added somewhat to the cost, which was $337 per acre. Even if duff had been more readily available, it is doubtful that this method could be employed as cheaply as the hay-wattle method.

The duff method was used on 12 less-exposed fills having a total area of 1.7 slope acres. Only three of these fills had northern exposures. Very good results were obtained and failures during the first winter occurred on only two fills, both having northern exposures. The areas showing damage were but 5 percent of the total area treated by this method.

On two low fills, special effort was made to encourage the growth of native plants. Material on an area having a rich growth of native grasses and flowers was reserved for these fills, and all duff, litter, humus and a small amount of topsoil were removed. This material was transported to the fills and spread by hand methods to a depth of about 1 inch. Detailed costs were not kept on this work, but estimated costs were less than $200 per slope-acre. These fills showed but slight erosion after the first winter, and a very good growth of native plants had been established by the following spring.

Route 74 was, to a large extent, experimental in the use of various types of treatment recommended by the Landscape Division of the Forest Service. In general the results obtained were satisfactory.

FIGURE 13.—PARTIAL FAILURE ON A FILL PROTECTED BY HAY WATTLES.

CONCLUSIONS

Fills protected by the various methods on the three projects were subjected to severe weathering during the winter of 1936–37. An inspection made in March 1937 revealed that the greater part of the work successfully prevented excessive erosion.

In view of the good results obtained by planting willow cuttings on route 20, it is thought that their use alone, without wattles, would prove as satisfactory as the combination.

On route 61, a comparison of the costs and percentages of failure for the three methods used indicates that method B (brush in fill layers with hay mat) has been the most satisfactory in the types of soil encountered.

While the soils found on route 61 washed to some extent, the major difficulty was their tendency to absorb and hold large quantities of water. The extreme weight of the saturated areas caused slides. The ideal treatment for such soils would probably be some form of waterproofing to prevent saturation. This would generally be economically impossible, and the only known alternative is to stabilize the saturated areas by providing some form of anchorage. Considerable stabilization can be attained by compacting fills to their extreme edges, and by incorporating all available rock in the fill slopes. Of the various forms of anchorage tried, the most efficient was that of placing rows of brush during fill construction. In spite of some failures of fills protected by this method, it is believed that satisfactory results could be obtained by spacing the brush layers closer and by using long-stemmed brush.

The types of work done on route 74 cannot be directly compared because the different methods were used on fills subjected to different weathering conditions. However, the results indicate that the methods can be

used as recommended with a fair assurance of satisfactory results on favorable types of soil.

Soils of the type and grading of those found on route 74 are subject to extreme erosion. These soils have little tendency to absorb and hold large quantities of moisture. Thus, while large slides do not occur on properly constructed fills, the surfaces ravel rapidly. The most suitable treatment for such soils is apparently the placing of some form of mat to hold the surface in place and prevent concentration of water in gullies. On low fills, where extensive concentration of water cannot occur, coverings of hay, straw, forest litter, or duff seem generally effective. On high fills anchorage of such coverings is necessary, and means of diffusing the water and preventing its concentration must be provided. Wattles and trenches, constructed in accordance with methods outlined in the Forest Service specifications, appear to be generally satisfactory for these purposes.

The results of the control work on these three projects have been of particular value in demonstrating the need of making thorough advance study of each proposed treatment and of adapting the methods to be used to each area treated. The fill soils should be studied to determine their reactions to the particular erosive forces to which they are exposed.

CHANNEL CHANGES ON FOREST HIGHWAYS

(Continued from page 175)

TABLE 2.— *Comparison of curvature data for project B (5.6 miles long) as originally surveyed, as revised, and as constructed*

Line	Curves	Total angle [1]	Maximum curve	Number of curves of—											
				1°	2°	3°	4°	5°	6°	7°	8°	10°	12°	14°	20°	22°
	Number	*Degrees*	*Degrees*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*
Original (1931)	29	1, 230	22			5	3	2	5	1	2	4	3	1	2	1
Revised (1934)	17	666	8	2	1	2	6			1	5					
Constructed	14	558	6	1	1	4	5	2	1							

[1] All curves on the constructed line are spiraled, and spiral angles are included in total angles for both the revised and the constructed line.

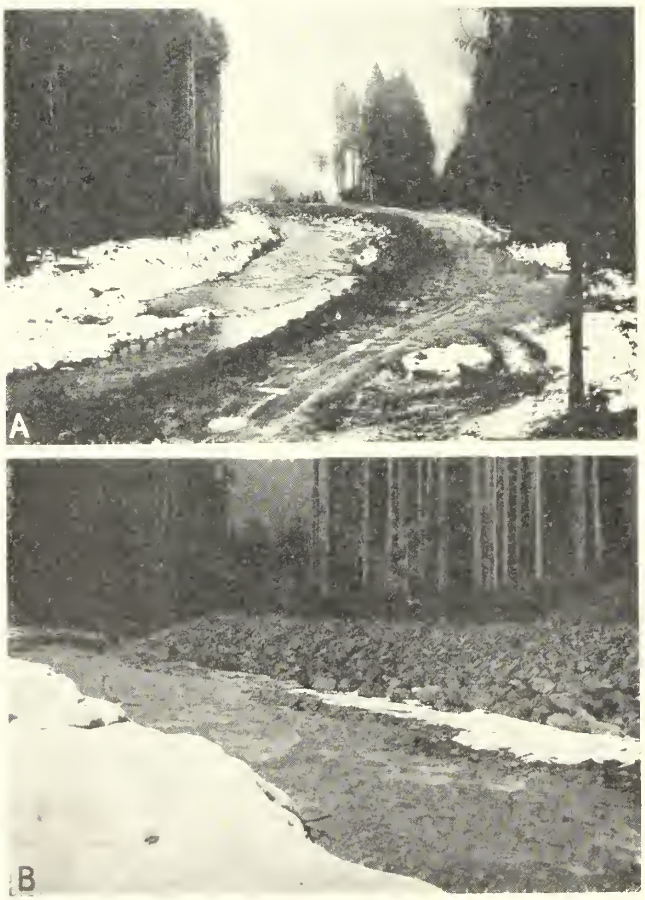


FIGURE 8.—TWO CHANNEL CHANGES: A, THIS CHANNEL CHANGE STRAIGHTENS A MEANDERING STREAM. THE RIPRAPPED SLOPE BEARS THE FULL FORCE OF THE CURRENT AROUND THE CURVE. B, THE LARGEST ROCKS ARE PLACED AT THE BOTTOM OF THE RIPRAPPED SLOPE.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF OCTOBER 31, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 163,700	$ 81,850	5.9	$ 2,557,661	$ 1,268,830	121.7	$ 3,108,900	$ 1,554,445	146.0	$ 4,942,055
Arizona	981,784	719,225	31.5	1,554,193	1,102,579	80.9	424,143	286,934	18.1	1,205,611
Arkansas	1,476,347	1,475,608	72.8	2,462,294	2,456,914	183.6	264,896	264,568	13.4	2,201,829
California	4,471,220	2,367,272	98.1	5,945,358	3,239,392	97.7	3,319,836	1,749,868	64.2	1,554,575
Colorado	2,234,049	1,248,194	87.4	1,260,853	696,448	33.9	494,404	260,018	16.0	2,166,805
Connecticut	710,998	353,096	8.7	231,110	107,923	1.3	72,610	36,300	.2	1,513,584
Delaware	231,150	115,530	5.8	281,162	140,537	13.7	243,905	119,934	14.8	1,107,687
Florida	301,250	150,620	8.2	2,716,172	1,358,086	65.2	508,170	254,085	7.8	2,648,148
Georgia	1,415,614	704,406	82.6	4,097,392	2,048,691	185.6	2,283,426	1,141,713	93.9	4,725,214
Idaho	1,639,071	978,378	140.6	1,271,274	759,487	78.3	194,020	115,570	20.3	1,112,436
Illinois	5,382,529	2,632,516	154.2	8,400,961	4,200,023	254.4	4,042,000	2,017,150	73.9	2,122,313
Indiana	2,527,473	1,258,071	53.3	5,091,782	2,545,770	148.9	1,206,198	603,099	24.4	2,120,799
Iowa	4,215,696	1,947,238	134.3	6,443,476	2,838,943	194.3	1,049,492	481,550	33.9	819,557
Kansas	2,575,957	1,260,617	146.6	3,694,085	1,846,943	125.7	1,900,812	949,133	98.1	3,276,079
Kentucky	904,102	452,051	25.6	3,914,808	1,907,404	115.2	1,845,164	922,582	88.0	2,401,247
Louisiana	213,312	102,519	5.9	6,724,240	1,514,885	48.6	5,293,838	836,455	12.5	1,729,722
Maine	1,316,680	658,340	34.7	2,143,566	1,071,783	52.7	690,599	345,295	16.3	203,604
Maryland	184,006	92,003	2.1	2,236,200	1,101,517	36.5	435,318	215,909	3.7	1,685,379
Massachusetts	926,641	463,320	7.3	4,039,654	2,019,826	15.2	1,484,726	742,363	.7	1,862,823
Michigan	4,829,650	2,414,825	126.1	5,882,030	2,941,515	145.4	2,417,309	1,133,829	37.3	228,478
Minnesota	3,861,230	1,919,018	175.3	3,782,371	1,879,941	164.3	1,528,426	628,328	59.6	1,741,332
Mississippi	804,200	402,100	40.0	4,696,100	2,347,730	216.5	1,362,200	680,050	58.3	3,203,296
Missouri	5,213,011	2,584,546	291.1	6,727,386	3,177,780	240.6	3,409,293	1,391,107	84.8	1,659,385
Montana	3,278,293	1,837,892	225.6	2,031,335	1,111,716	112.3	255,860	143,920	7.9	2,347,702
Nebraska	1,757,862	878,931	169.2	4,807,736	2,391,846	465.8	2,359,810	549,905	64.6	2,311,049
Nevada	1,333,105	1,146,790	74.9	1,454,212	1,255,676	61.5	328,489	284,859	36.5	682,069
New Hampshire	143,483	71,379	1.9	595,203	293,799	11.2	56,074	27,911	.6	1,039,780
New Jersey	689,590	267,500	5.7	1,808,138	903,749	16.5	124,870	62,435	.6	2,247,469
New Mexico	2,141,540	1,315,548	169.9	2,447,626	1,592,186	125.0	792,104	483,101	35.7	254,189
New York	6,833,274	3,162,558	119.4	17,878,354	8,498,916	300.5	3,583,890	1,790,445	49.3	565,736
North Carolina	2,086,699	1,043,289	163.1	6,076,383	2,825,491	300.2	1,592,280	697,390	61.3	2,538,696
North Dakota	925,760	925,760	173.4	1,281,430	1,260,920	90.5	236,822	236,822	36.4	3,298,731
Ohio	1,796,762	847,903	24.5	10,191,439	5,046,014	112.2	2,020,460	1,010,230	25.8	5,183,848
Oklahoma	2,119,328	1,112,698	90.7	2,829,097	1,464,273	118.5	1,859,295	976,663	82.1	3,123,267
Oregon	2,599,477	1,517,777	111.1	2,172,633	1,325,099	60.1	648,679	395,770	30.5	1,045,089
Pennsylvania	7,093,759	3,527,559	106.9	9,094,345	4,527,953	120.0	4,238,870	2,109,206	67.5	2,703,665
Rhode Island	536,250	266,814	5.7	1,097,274	548,637	13.7	257,230	128,615	1.7	760,655
South Carolina	1,927,691	820,202	167.2	5,068,911	2,049,055	229.7	1,694,090	630,135	71.1	1,357,311
South Dakota	1,853,022	1,057,041	148.0	1,905,350	1,053,580	214.0	549,730	303,940	83.5	3,132,700
Tennessee	932,278	459,194	42.7	1,539,764	769,882	41.5	803,300	401,650	29.5	4,888,945
Texas	6,730,726	3,362,428	486.9	10,373,623	5,168,332	525.0	3,988,591	1,945,434	183.6	5,862,276
Utah	491,434	353,242	44.6	1,445,090	1,030,780	130.1	154,270	108,800	8.5	1,096,863
Vermont	664,181	322,733	18.1	1,594,810	719,760	43.3	302,650	127,954	5.5	82,385
Virginia	1,736,001	868,001	90.6	2,829,382	1,375,276	78.6	1,151,502	551,316	40.2	2,125,909
Washington	1,266,828	665,300	62.1	2,871,529	1,500,679	44.9	1,007,410	527,225	8.9	796,634
West Virginia	708,835	354,418	20.5	1,466,674	736,830	42.7	628,902	464,851	18.1	2,136,392
Wisconsin	6,140,018	2,888,797	202.9	5,150,820	2,519,820	136.4	775,055	384,600	31.4	1,309,076
Wyoming	2,154,791	1,318,461	203.9	1,426,942	863,629	189.9	246,250	152,150	12.4	218,942
District of Columbia	459,087	226,420	8.0	504,649	248,452	9.9	449,816	205,089	6.9	1,148,247
Hawaii				53,980	26,990	2.1	648,039	322,040	12.6	275,970
Puerto Rico										
TOTALS	104,979,744	54,999,978	4,675.6	186,150,857	93,682,287	6,216.3	68,334,023	31,752,751	1,998.9	98,765,553

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF OCTOBER 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,944,942	$ 3,907,392	136.9	$ 250,100	$ 243,478	7.8				$ 245
Arizona	2,569,841	3,125,360	2,453,294	194.8	38,548	38,548					77,999
Arkansas	3,352,061	3,143,188	3,123,140	350.5	181,216	180,523	9.7				48,398
California	7,747,928	7,545,686	7,194,317	257.5	572,929	515,656	6.4				37,955
Colorado	3,395,263	2,379,345	2,294,218	99.4	89,597	89,596	6.0	$ 8,200	$ 8,200		1,003,249
Connecticut	1,418,709	1,099,482	1,030,861	16.2	306,181	280,480	6.2	124,130	64,435	0.2	42,933
Delaware	900,310	627,224	601,492	53.8	257,585	257,585	13.0	26,712	26,712	4.4	14,521
Florida	2,597,144	2,584,572	2,521,530	99.1	38,957	38,957					36,657
Georgia	4,988,967	1,282,346	1,245,396	81.4	2,694,894	2,426,514	118.8	684,630	684,630	34.5	632,427
Idaho	2,222,747	2,252,832	2,151,909	185.6	49,205	49,131	.3				21,707
Illinois	8,694,009	8,034,975	7,803,546	456.2	764,590	764,590	32.0				125,873
Indiana	4,941,255	4,854,405	4,581,943	219.7	331,419	331,419	18.7				27,893
Iowa	4,991,664	5,218,382	4,865,314	526.3	124,300	117,465	2.3	3,474	3,470		5,415
Kansas	4,994,975	4,580,650	4,499,689	356.3	419,276	417,103	34.2	15,620	15,620	1.7	62,562
Kentucky	3,726,271	3,334,467	3,183,269	348.3	459,839	459,839	9.6	57,000	57,000	.6	26,163
Louisiana	2,890,429	2,473,382	2,212,488	163.0	604,151	543,516	4.6	97,087	97,070	10.4	37,355
Maine	1,676,799	1,573,431	1,560,841	72.0	118,557	115,958	4.1				
Maryland	1,750,738	637,876	631,089	23.5	581,095	581,095	11.6	193,134	159,302	6.4	379,253
Massachusetts	3,262,885	1,103,580	1,103,580	14.0	1,910,026	1,519,256	4.4	1,136,726	568,363	.8	71,686
Michigan	6,301,414	6,518,192	5,952,637	288.6	290,686	289,571	3.3	44,500	33,882	.3	25,324
Minnesota	5,277,145	6,384,622	5,170,637	895.7	111,965	92,862	6.1				13,646
Mississippi	3,457,552	2,831,756	2,827,119	197.9	541,746	540,706	37.8	8,900	8,900	.6	80,826
Missouri	6,012,652	5,298,465	5,157,460	774.5	802,370	747,248	3.2	34,391	32,294		75,650
Montana	3,676,416	3,618,384	3,562,698	202.6	95,385	95,385	.1	8,462	8,462		9,872
Nebraska	3,870,739	3,311,368	3,205,529	346.4	587,690	587,686	24.3	70,270	70,270	1.8	7,254
Nevada	2,243,074	2,290,966	2,207,403	110.1	33,646	33,646					2,026
New Hampshire	945,225	876,074	843,821	37.7	63,555	63,555	2.5	57,537	37,746	3.2	103
New Jersey	3,129,805	1,195,787	1,177,568	20.5	1,917,769	1,917,769	14.9				34,468
New Mexico	2,871,397	2,751,452	2,746,286	205.3	106,210	106,210	8.4	14,681	12,196		6,705
New York	11,046,377	10,786,792	10,331,736	170.0	125,700	85,700	.6	40,000	40,000		588,941
North Carolina	4,720,173	4,020,158	3,949,305	259.4	716,990	716,390	31.3	37,900	37,900	.5	16,577
North Dakota	2,867,245	2,490,916	2,459,240	378.9	107,799	107,799	1.2	280,295	280,295	36.0	19,911
Ohio	7,670,815	6,150,464	6,064,711	261.1	1,571,192	1,535,767	35.6	30,200	30,200	1.9	40,137
Oklahoma	4,580,670	4,329,543	4,202,267	391.5	330,110	330,110	13.7	77,400	35,307	3.8	12,986
Oregon	3,038,642	3,127,157	2,861,791	161.0	134,854	134,854	3.6	11,846	11,846	1.0	30,151
Pennsylvania	9,347,797	5,355,490	5,119,017	215.8	3,846,235	3,444,595	50.7	785,495	630,267	19.3	153,918
Rhode Island	989,208	1,111,600	989,136	18.8							72
South Carolina	2,702,012	2,261,423	2,111,864	224.6	627,419	545,752	24.9	9,664	9,664	1.5	34,731
South Dakota	2,976,454	2,532,822	2,529,342	467.0	432,947	432,947	37.6				14,165
Tennessee	4,192,460	3,173,312	3,141,998	126.1	1,020,913	1,020,913	24.6				29,549
Texas	11,989,350	12,599,090	11,528,578	1,105.8	382,509	285,584	5.9	159,161	143,575	13.3	31,614
Utah	2,067,154	2,053,065	1,842,753	204.2	196,775	196,775	4.5				27,626
Vermont	924,306	1,057,603	911,448	23.2	8,000	8,000					4,858
Virginia	3,652,667	3,345,780	3,264,377	1,020.5	188,931	182,932	14.6				205,358
Washington	3,026,161	3,356,252	2,940,122	164.3	45,392	45,392		25,465	20,845	.7	19,802
West Virginia	2,231,412	1,482,752	1,473,104	60.5	854,026	718,128	34.6	47,560	40,180	5.4	
Wisconsin	4,823,884	5,244,297	4,729,965	343.4	94,268	93,900	.3				19
Wyoming	2,219,155	2,185,929	2,180,273	152.4	33,287	33,287					5,595
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	678,560	655,700	10.4	277,293	215,689	7.0	10,000	10,000		44,644
TOTALS	195,000,000	173,166,196	164,052,689	12,501.5	25,338,127	23,579,861	681.0	4,100,440	3,178,631	148.3	4,188,819

U. S. GOVERNMENT PRINTING OFFICE: 1937

Vol. 18, No. 9

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Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF OCTOBER 31, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,249,498	$ 3,231,948	44	1	12	$ 670,572	$ 670,572	5			$ 175,300	$ 109,000	4			$ 23,097
Arizona	1,256,099	1,113,686	1,072,656	13			185,468	163,059	2		3	6,403	6,403			2	13,981
Arkansas	3,574,060	2,826,690	2,819,516	51	6	4	570,659	570,224	4		8	149,214	148,944	1		21	35,376
California	7,486,362	7,153,440	6,966,322	45	8		470,205	470,204	2			10,000	10,000	5			39,836
Colorado	2,631,567	1,899,531	1,837,167	23	1		821,674	791,106	7	2	21						3,294
Connecticut	1,712,624	297,379	297,379	2	1	1	1,341,801	1,320,440	8								94,865
Delaware	418,239	130,000	130,000	1			277,993	277,993	2								10,246
Florida	2,827,883	2,271,833	2,252,939	28	5	2	250,124	249,518	3			128,320	128,320			72	197,106
Georgia	4,335,343	355,157	353,665	11	5	4	1,524,041	1,524,041	27	6	7	521,070	521,070	7	2	41	2,497,173
Idaho	1,674,479	1,333,866	1,323,563	20	2	9	318,696	318,661	3		9	4,261	4,261			3	27,994
Illinois	10,307,184	8,062,094	7,986,847	62	8		2,212,294	2,205,890	11	2		127,000	78,000	2			36,447
Indiana	5,111,096	3,899,953	3,726,047	35	12		1,317,906	1,317,906	7		162						67,143
Iowa	5,600,679	4,362,927	4,263,256	94	9	8	1,332,589	1,326,781	12								10,642
Kansas	5,246,258	3,966,373	3,904,919	54		5	1,273,119	1,253,704	4	1		108,004	15,000	2			72,634
Kentucky	3,672,387	1,305,853	1,298,000	18	5		2,037,014	1,747,282	6	6		612,579	612,579	3	1		14,526
Louisiana	3,213,467	1,328,350	1,328,343	14	1		1,120,878	1,120,859	11	1		657,841	608,350	4	1		155,315
Maine	1,426,861	1,069,189	1,066,911	19		2	373,818	355,570	1	2		4,380	4,380			2	
Maryland	2,061,751	501,739	501,739	3	3	16	847,170	847,170	6	2	3	480,361	407,893	2	1	11	304,949
Massachusetts	4,210,833	2,139,608	2,137,178	19	4		1,617,761	1,617,761	7			249,991	249,991	1			205,903
Michigan	6,765,197	6,472,627	6,154,021	43	8		485,477	485,477	1			43,500	43,500		1		82,199
Minnesota	5,395,441	4,371,941	4,267,794	78	14	50	1,115,218	1,104,074	8								23,573
Mississippi	3,241,475	2,038,818	2,038,651	45	4	4	689,238	689,238	10	3	4	76,700	76,700	2		13	436,886
Missouri	6,142,153	3,014,620	2,940,676	31	1		3,293,571	3,190,761	18		1	1,650	1,650			1	9,066
Montana	2,722,327	2,589,093	2,532,574	37	7		245,576	185,931	1	1							3,823
Nebraska	3,556,441	2,651,081	2,615,327	73	3	19	713,266	713,266	8		1	214,916	189,878	5		4	37,971
Nevada	887,260	885,084	857,417	8	3		13,308	13,308				3,630	3,630			5	12,905
New Hampshire	822,484	737,902	737,857	8	4	6	106,670	82,521	2			2,106	2,106			1	
New Jersey	3,983,826	2,806,217	2,794,784	18	5	1	1,084,370	1,084,370	6	4		44,730	44,730		1		59,942
New Mexico	1,725,286	1,644,472	1,637,584	18	1		82,384	82,384	1								5,318
New York	13,577,189	10,043,525	9,769,001	31	39		3,537,000	3,537,000	14	10		94,000	94,000				177,189
North Carolina	4,823,958	3,297,724	3,277,598	45	18		1,405,139	1,405,139	15		108	32,870	32,870	2			108,350
North Dakota	3,207,473	2,820,675	2,817,055	56	4		389,012	389,012		1							1,406
Ohio	8,439,897	1,791,055	1,681,905	12	3	2	6,205,013	5,726,074	35	5	1	868,700	853,700	12		11	178,218
Oklahoma	5,004,711	3,582,337	3,575,085	56	7	4	1,436,634	1,303,534	9	5	3	102,170	102,170			34	23,922
Oregon	2,334,204	2,239,057	2,154,389	15	6	2	164,842	161,937	2								17,878
Pennsylvania	11,483,613	6,555,714	6,042,786	60	17	9	5,370,140	5,003,152	26	6		150,473	150,473	5			287,202
Rhode Island	699,691	698,074	695,860	4	3												3,831
South Carolina	3,059,956	1,500,273	1,481,282	29	10	8	1,074,226	1,066,796	16	5	22	205,667	205,667	1	1	18	306,211
South Dakota	3,249,086	2,263,266	2,262,574	45	6	53	770,289	770,289	20			269,350	210,392	17			5,831
Tennessee	3,903,979	1,305,258	1,296,577	25	3	26	2,201,440	2,201,440	20	1	1	155,070	155,070	1		12	250,892
Texas	10,855,982	9,774,780	9,767,329	125	14	65	560,011	548,167	2	2	9	223,121	223,121			75	317,365
Utah	1,230,763	931,508	924,612	12	1	1	290,423	290,423	5								15,728
Vermont	729,857	749,776	705,709	10	7	18	14,100	14,100			2						10,048
Virginia	3,774,287	2,493,195	2,383,324	42	15	14	1,340,098	1,333,473	10	5	2	49,660	47,646		1	8	9,843
Washington	3,095,041	2,733,615	2,710,650	22	11	11	368,494	368,069	1	1							16,322
West Virginia	2,677,937	749,099	749,099	6	1	4	1,847,604	1,845,636	20	2	2	78,855	78,855	2		1	4,348
Wisconsin	5,022,683	4,019,300	3,989,778	37	5		973,618	973,618		2		109,723	56,925			28	2,362
Wyoming	1,360,841	1,165,661	1,156,283	12			189,365	189,365	2								15,193
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			226,162	158,370	2								11,327
TOTALS	196,000,000	133,895,583	131,210,785	1565	291	360	54,756,470	53,065,665	382	75	369	5,961,615	5,477,274	78	9	363	6,246,276

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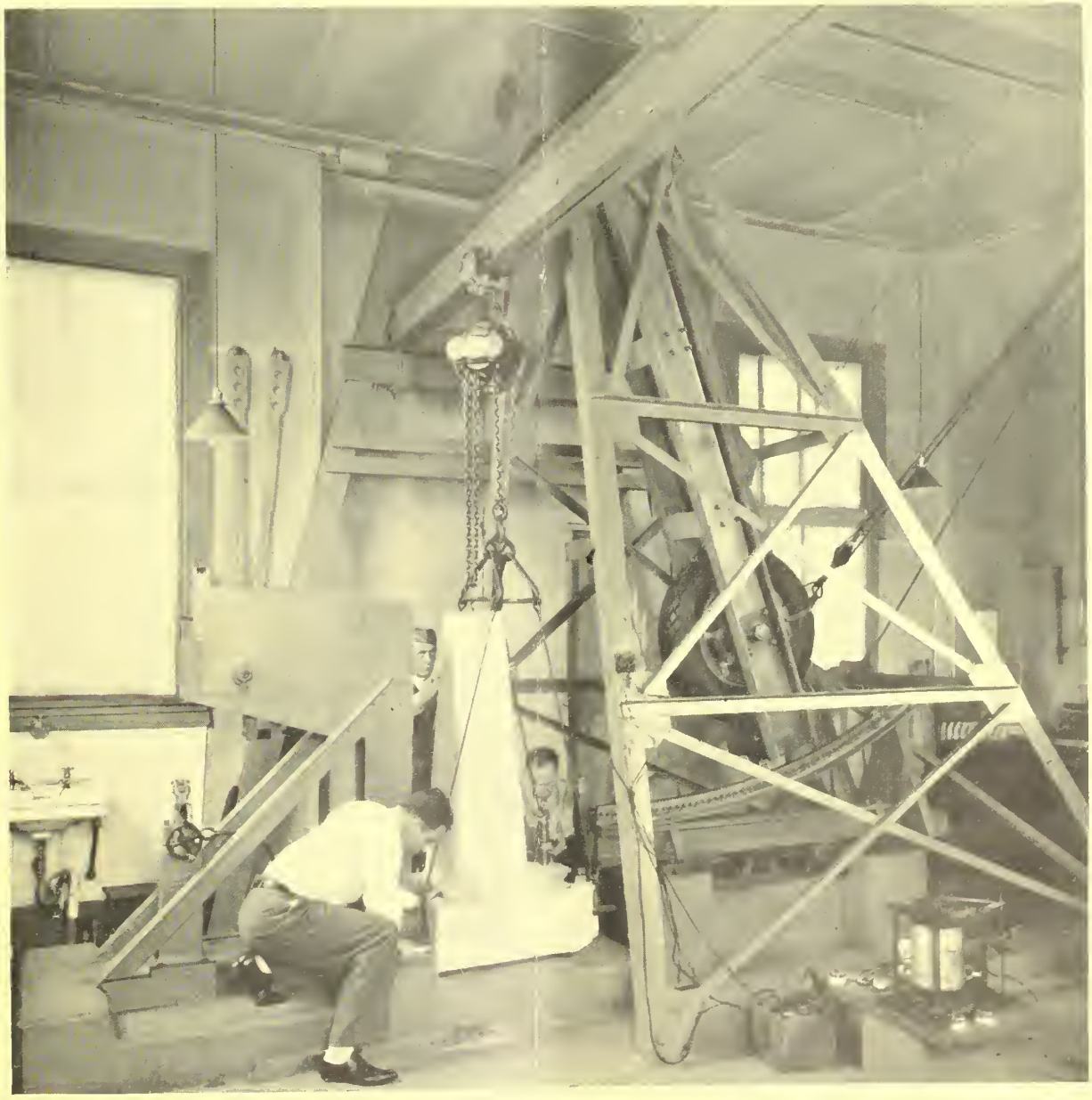
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 10 DECEMBER 1937



MACHINE FOR MAKING IMPACT AND SUSTAINED LOAD TESTS OF CONCRETE

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 10 December 1937

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

A MACHINE FOR IMPACT AND SUSTAINED LOAD TESTS OF CONCRETE

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Senior Engineer of Tests, and JAMES A. BUCHANAN, Associate Engineer of Tests

ONE of the fields of research in which the Bureau of Public Roads has been engaged for a number of years is that concerned with the structural action road surfaces. The development of a knowledge of the forces to which road surfaces are subjected and of the manner in which pavements react to those forces are the two major objectives in this research.

An essential part of this general investigation has been a study of the forces to which road surfaces are subjected by vehicle wheels. These forces may be of a sustained nature if the vehicle is stationary; they may be of a transient nature if the vehicle is rolling smoothly over the pavement; or they may occur quite suddenly where irregularities in the surface contour of the road induce impact reactions.

Load limitations imposed by law, and data on actual vehicle weights obtained in numerous traffic surveys, indicate the order of magnitude of the forces to which pavements are likely to be subjected by the wheels of static or slowly moving vehicles. The extensive studies of motor-vehicle impact phenomena made in this country have developed quite definite knowledge of the reactions to be expected under various conditions and of the influence of each of the major factors that determine the magnitude.[1] More recently work along similar lines has been conducted in England and the published data from these researches[2] are in general accord with those obtained in the studies mentioned above.

It can be said, therefore, that so far as the structural action of pavements is concerned, there exists a fairly adequate knowledge of the forces which may be expected from the wheels of present-day vehicles.

Of equal importance is the development of a knowledge of the effects of these forces on pavement structures of various types. Both theoretical analyses[3] and experimental researches[4] have furnished a considerable amount of fundamental information concerning the effects of static or slowly applied forces on pavements of the rigid type, out of which there is evolving a better understanding of the principles of design for such pavements for loads of this nature. When, however, the effects of suddenly applied forces (such as impact reactions) are considered, the information available is much less satisfactory. In spite of the attention that has been given in recent years to the impact testing of materials of all sorts, the fact remains that the underlying principles of stress development under impact are not well understood.

TEST METHOD SHOULD MEET SEVERAL REQUIREMENTS

It is generally agreed that static and impact forces may differ in their effects and it is believed that the degree of difference depends upon the time rate of deformation in the material being stressed, and probably upon other factors. However, the extent to which the various phenomena apply in the case of motor-vehicle impact has not been known.

The approach to a study of the effects of impact forces on rigid pavements is somewhat simplified by two facts: First, portland cement concrete is the principal material to be considered; and second, the total elapsed time of the impact reaction varies from about 0.05 to 0.15 second, which is neither very short nor is the range great as compared with those encountered in other fields. The absolute maximum force developed during any impact reaction is, of course, instantaneous.

As a first step in its program of research on this particular subject the Bureau of Public Roads undertook a thorough study, under controlled conditions in the laboratory, of the behavior of large-size flexure specimens of concrete when subjected to sustained and to impact forces of comparable magnitudes. This work has now been in progress for about 2 years and many tests have shown the method of test and the testing equipment to be quite satisfactory. It is the purpose of this paper to outline the method of test that was adopted and to describe the testing machine that was designed and built for this purpose, a machine that is unique in several respects. The presentation of the results of the tests that are being made will be left to a subsequent discussion.

Since the object of the study was to determine the relative effects of sustained and impact forces on the flexural behavior of concrete, the selection of a method of test and the design of the testing machine were guided by the following basic requirements:

1. It should be possible to apply both sustained and impact forces of known magnitudes to a specimen in such a manner that the only variable present would be the duration of the applied force.
2. The specimens should be flexure specimens of portland cement concrete.
3. The magnitude of the forces, the duration of the forces, the masses involved in the impact, the flexing of the concrete and other conditions surrounding the test, should not depart too widely from conditions that obtain when a pavement slab sustains a wheel load.
4. The testing machine must be of a practicable size.
5. The force system should be determinate.

A consideration of these requirements led to the adoption of the method of test shown schematically in

[1] The Motor Truck Impact Tests of the Bureau of Public Roads, by Earl B. Smith, PUBLIC ROADS, vol. 3, No. 35, March 1921.
Motor Truck Impact as Affected by Tires, Other Truck Factors and Road Roughness, by James A. Buchanan and J. W. Reid, PUBLIC ROADS, vol. 7, No. 4, June 1926.
Motor Truck Impact as Affected by Rubber Tread Thickness of Tires, by James A. Buchanan, PUBLIC ROADS, vol. 11, No. 7, September 1930.
Road Impact Produced by a Heavy Motor Bus, by James A. Buchanan, PUBLIC ROADS, vol. 13, No. 9, November 1932.

[2] Impact of Wheels on Roads, by Aughtie, Batson, and Brown. Proceedings, Institution of Civil Engineers (Br.), vol. 237, Session 1933-34, part I.
The British researches are also reported in the Reports of The Ministry of Transport, Roads Department, beginning with that for the year 1930.

[3] Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, PUBLIC ROADS, vol. 7, No. 2, April 1926; and Analytical Tools for Judging Results of Structural Tests of Concrete Pavements, by H. M. Westergaard, PUBLIC ROADS, vol. 14, No. 10, December 1933.

[4] Stress Measurements in Concrete Pavements, by L. W. Teller, Proceedings, Fifth Annual Meeting, Highway Research Board, December 3-4, 1925.
The Six-Wheel Truck and the Pavement, by L. W. Teller, PUBLIC ROADS, vol. 6, No. 8, October 1925.
The Structural Design of Concrete Pavements, by L. W. Teller, and Earl C. Sutherland, PUBLIC ROADS, vol. 16, Nos. 8, 9, and 10, and vol. 17, Nos. 7 and 8.

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figure 1. A compound pendulum, carrying a vehicle wheel at its center of percussion, is used to apply horizontal forces to a test specimen that is arranged as a vertical cantilever. Impact forces are developed by swinging the pendulum, and sustained forces are developed by exerting a pull on the axle of the wheel on the pendulum.

It is readily apparent that this arrangement can, by proper design, be made to satisfy the basic requirements stated above. Either sustained or impact forces can be applied to the specimen at the same point and through the same contact medium, the tire. The horizontal application of forces is advantageous, as the gravitational effect is eliminated; and, in the case of the impact force, a single blow without secondary rebound can be applied. The cantilever specimen has definite advantages and the use of a beam of varying width gives a section that has useful stress variation characteristics and simulates to a fair degree the unsupported corner of a pavement slab. The characteristics of the pendulum as a dynamic machine can be theoretically established and empirically checked. By designing a machine and specimen of suitable proportions, the forces and masses involved in the test reach satisfactory values.

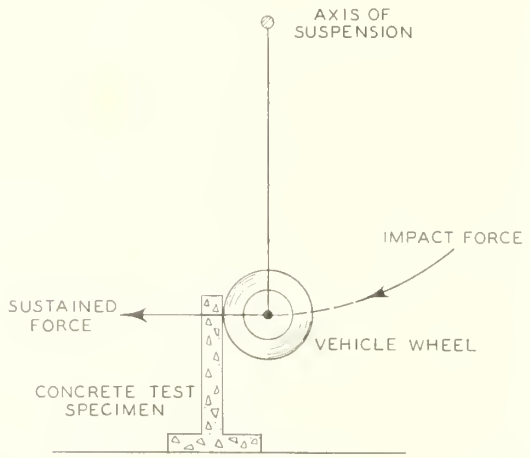


FIGURE 1.—SCHEMATIC DIAGRAM SHOWING THE PRINCIPLES OF THE TEST.

The design of the test specimens determined the arrangement of the testing machine and certain of its principal dimensions. It is logical, therefore, to describe these specimens before discussing the machine itself.

FOUR TYPES OF TIRE SELECTED FOR TESTING IN MACHINE

The general form of the test specimens is shown in figure 2. Except for the thickness of the vertical cantilever the dimensions of all specimens are the same. The cantilever beams are 48 inches long, measured from the top surface of the heavy integral base to the free end. The cantilever portion is 30 inches wide at the base, 5 inches wide at the top or free end, and of a constant thickness. Specimens 4, 6, 8, 10, and 12 inches thick have been made. The integral base portion used to anchor the test specimen to the floor is 10 inches thick. No reinforcing steel has been used in any of the specimens.

The center of the area of force application is 40 inches above the base, at which point the facial area of the beam is sufficient to accommodate the entire area of contact for any of the tire and load conditions that it is planned to use.

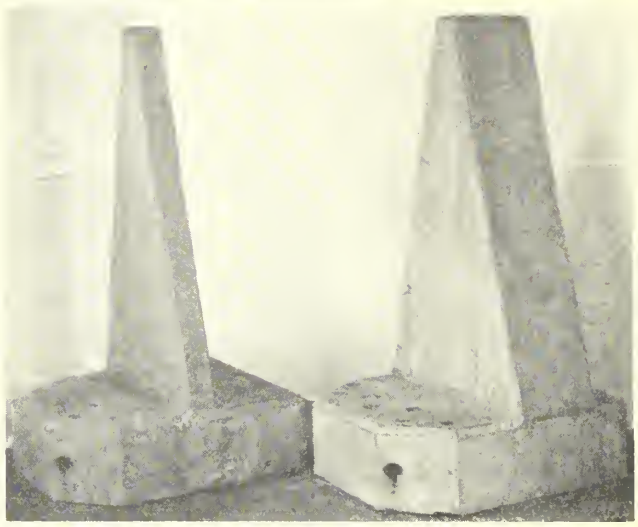


FIGURE 2.—CANTILEVER TEST SPECIMENS. THIS VIEW SHOWS THE CONTRAST BETWEEN SPECIMENS 4 AND 12 INCHES THICK AND ALSO SHOWS A NUMBER OF DETAILS TO WHICH REFERENCE IS MADE LATER.

Because of their direct effect on the design of the machine, the probable maximum bending stress and the probable deflection of the cantilever at the center of the area of force application were calculated for each of the five thicknesses of specimen. These calculated values for a range of forces are shown in figure 3. The values indicate that a maximum force of about 15,000 pounds would probably be required, assuming concrete of normal strength. A large percentage of the testing, however, would involve forces of 6,000 pounds or less.

The range in forces required for testing the beams indicated that appropriate tire equipment would have a normal or rated load-carrying capacity of about 2,000 pounds. In order to make it possible to vary the durations of the impact reactions and also the areas of contact over which the forces were applied to the test specimen, interchangeable tires of the four types commercially available were chosen. In order that the effect of tread design variations might be minimized, tires with a uniform tread design were obtained. The final selection of actual sizes was made in conformity with the recommendations of the Tire and Rim Association, for truck and bus equipment, in effect at the time of purchase and are described as follows:

Solid—34 by 4 inch, low profile, solid center, capacity 1,900 pounds.

Cushion—36 by 5 inch, high profile, hollow center, capacity 1,900 pounds.

Pneumatic—32 by 6 inch, 10 ply, high pressure, capacity 1,900 pounds at 78 pounds per square inch inflation pressure.

Balloon—7.50/20 inch, 8 ply, low pressure, capacity 1,900 pounds at 49 pounds per square inch inflation pressure.

The general load deformation characteristics of the tires were determined by sustained load tests in which the radial deflections were measured and areas of contact on a plane surface were obtained. A smooth envelope drawn around the tire imprint determined the length, width, and gross area of contact. The results of these tests are shown in figure 4. The tires were put in a "cyclic state" by repeated loadings before obtaining the test values for this figure. The inflation pressures for the two pneumatic tires were measured with the tires free from applied load.

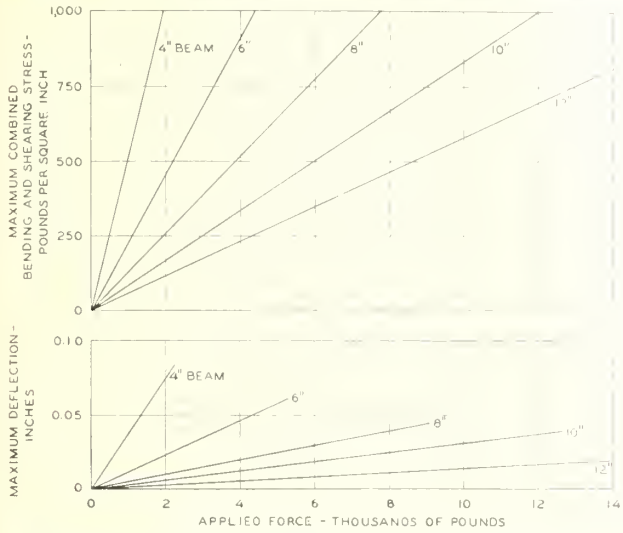


FIGURE 3.—MAXIMUM STRESSES AND DEFLECTIONS OF CONCRETE CANTILEVER TEST SPECIMENS.

In the case of the impact test by this method the effective weight of the pendulum[5] corresponds to the unsprung component of the static wheel load of a vehicle. Provided the effective pendulum weight is not unduly out of proportion to the unsprung weight ordinarily encountered in vehicles equipped with tires of 1,900 pounds capacity, it is not necessary to restrict the pendulum mass to a particular value, so long as this mass is sufficient to develop the desired maximum force with a reasonable pendulum length and arc of swing. These requirements led to an early tentative selection of an effective weight which was of the order of one third of the tire capacity.

Before proceeding further with the practical aspects of the design it is desirable to review briefly the elementary theory of the pendulum, as it has a bearing upon much of the discussion that follows.

MATHEMATICAL RELATIONSHIPS FOR COMPOUND PENDULUM OUTLINED

If a heavy mass could be concentrated at a point and suspended by a perfectly flexible and weightless cord it would constitute what is known as a simple pendulum. When such a pendulum oscillates or swings in a vertical plane it behaves according to definite physical laws. The velocity, acceleration, kinetic energy, and other properties, may be exactly determined. For example, the period or time of oscillation of the simple pendulum is known to depend upon (1) the length of the pendulum, (2) the force of gravity, and (3) the angle of swing according to the following relation:

$$T=2\pi\sqrt{\frac{l}{g}\frac{\theta}{\sin\theta}}$$

in which

T=the time required for one complete oscillation, in seconds.
l=the length of the pendulum.
g=the acceleration of gravity.
θ=the angular displacement from the vertical, in radians.

It is physically impossible to construct a simple pendulum. Frequently an actual pendulum is built up of a number of parts, rigidly connected. Each particle of this composite whole, being at a fixed distance from the axis of support, tends to oscillate as a simple pendulum in its own natural period. The rigid connection with other particles prevents this, and all of the particles that constitute the composite structure are forced to oscillate in a common period. Some of the particles will oscillate more rapidly, others more slowly than their natural periods would demand. However, there will be some whose natural period will be the same as that of the composite mass and the distance from the axis of support to these particles will be the length of the equivalent simple pendulum, that is, the hypothetical pendulum having the same period of oscillation.

The point at which the particles in the rigid or compound pendulum, as it is called, are oscillating in their own natural period is known as the center of oscillation because it is interchangeable with the center of suspension without affecting the period of the pendulum. An interesting and important property of the pendulum is that if it is struck a blow at the center of oscillation, normal to the pendulum and in the plane of oscillation, rotation will be produced about the axis of suspension but the blow will create no force reaction at this axis. For this reason the center of oscillation is often referred to as the center of percussion.

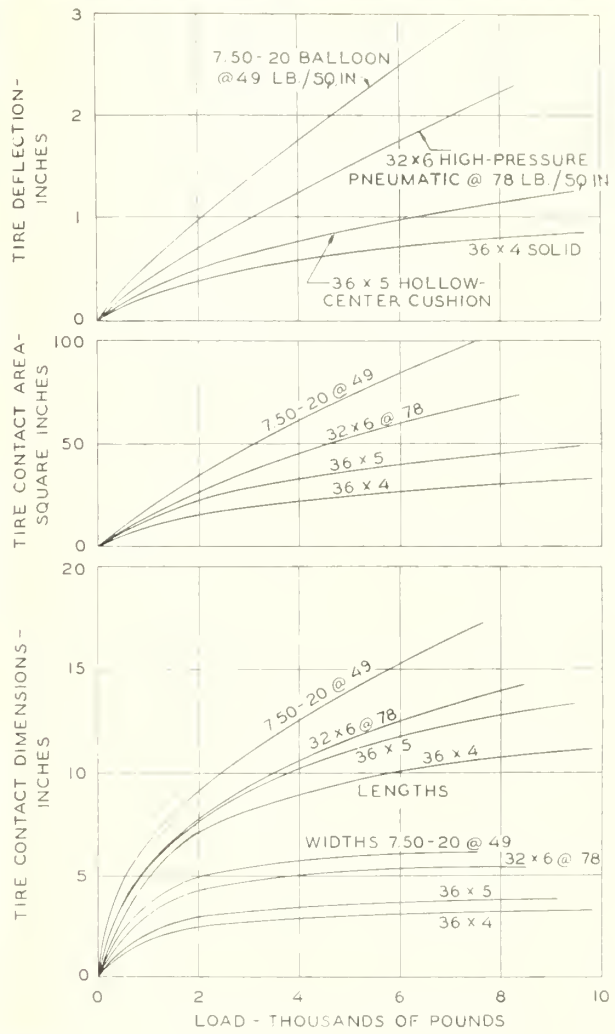
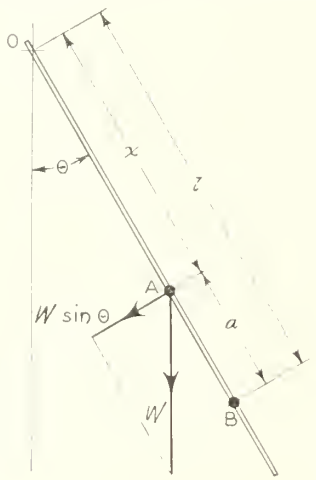


FIGURE 4.—CHARACTERISTICS OF TIRES UNDER SUSTAINED LOADS.

[5] By effective weight is meant that part of the total pendulum weight which is used in calculating the mass effective in developing force against the specimens during impact.

Because a compound pendulum does not conform to the defined requirements of a simple pendulum, its motion cannot be predicted directly from the laws of the simple pendulum. These laws must be modified to satisfy both the laws of an oscillating body and those of a rotating body. As such, they apply to the equivalent simple pendulum. It is with a compound pendulum that the present discussion is concerned and the principles of its motion are those to be considered.



O - CENTER OF SUSPENSION
A - CENTER OF GRAVITY
B - CENTER OF OSCILLATION OR PERCUSSION
x AND a - DISTANCES FROM THE CENTER OF GRAVITY TO "O" AND "B", RESPECTIVELY
l - LENGTH OF EQUIVALENT SIMPLE PENDULUM
W - WEIGHT OF PENDULUM
θ - ANGLE OF DISPLACEMENT FROM THE VERTICAL

FIGURE 5.—DIAGRAM OF THE COMPOUND PENDULUM.

Referring to figure 5, which shows in diagrammatic form the essentials of a compound pendulum, it is apparent that the position of the center of gravity, A, can be determined by taking the moments of the various individual masses which go to make up the total weight, W, about the center of suspension, O, and dividing their summation by this weight

$$x=\frac{\Sigma M_0}{\Sigma W}$$

The moment of inertia of the pendulum about O, ΣI_0, is determined in a similar manner by taking the moments of inertia of the individual masses about O, and adding them.

The radius of gyration, k_0, may be determined from the moment of inertia and weight of the pendulum by dividing, as follows:

$$k_0=\sqrt{\frac{\Sigma I_0}{\Sigma W}}$$

The distance, l, from the center of suspension, O, to the center of oscillation (or percussion), B, may then be determined by the following relation

$$l=\frac{k_0^2}{x}$$

This, it will be remembered, is the length of the equivalent simple pendulum, a pendulum having the same natural period as the compound pendulum. This period, as previously stated, is determined by the formula

$$T=2\pi\sqrt{\frac{l}{g}\frac{\theta}{\sin\theta}}$$

In connection with the use of this formula the value of the term $\frac{\theta}{\sin\theta}$ is of interest. It is shown in table 1 for several angles within the range under consideration.

TABLE 1.—*Values of* $\frac{\theta}{\sin\theta}$ *for several angles*

Value of angle θ		Sin θ	$\frac{\theta}{\sin\theta}$	$\sqrt{\frac{\theta}{\sin\theta}}$
Degrees	Radians			
0° 30′	0.008727	0.008727	1.0000	1.0000
1°	.017453	.017452	1.0001	1.0000
2°	.034907	.034900	1.0002	1.0001
3°	.052360	.052336	1.0005	1.0002
10°	.174533	.173648	1.0051	1.0025
20°	.349066	.342020	1.0206	1.0102
30°	.523599	.500000	1.0472	1.0233

CENTER OF PERCUSSION AND CENTER OF FORCE APPLICATION MADE TO COINCIDE

These values indicate that in computing the period of oscillation this term may be omitted, if the angle of swing is small, for all but the most precise determinations.

If the sum of the moments about O is divided by the length, l, the resulting value is the weight to be used in computing the mass effective at the center of percussion during impact. This effective weight, converted to a mass value, and multiplied by the deceleration [6] developed when the motion of the pendulum is reversed, gives the dynamic force of the reaction.

The angular velocity, ω, of the oscillating pendulum may be obtained from its period of oscillation by the expression

$$\omega=\frac{2\pi}{T}=\sqrt{\frac{g}{l}\frac{\sin\theta}{\theta}}$$

The maximum tangential velocity, v_t, of the center of percussion (at the bottom of the swing) is

$$v_t=\omega\theta l$$

the value of θ being expressed in radians.

The angular acceleration, α, similarly stated, is

$$\alpha=-\frac{gx\sin\theta}{k_0^2}\text{ or }-\frac{g\sin\theta}{l}$$

the tangential acceleration at the center of percussion a_t, being

$$a_t=-g\sin\theta.$$

Having decided upon the type and sizes of the test specimens, upon the types and sizes of the tires, and upon the general order of mass required in the pendulum, there still remained other and more detailed requirements to be considered in the design, for example:

The center of percussion of the pendulum and the center of the area of force application on the specimen

[6] Acceleration is defined as the second derivative of the displacement-time relation, or as the time rate of change of velocity. As such it must be either positive or negative. Deceleration is a term frequently employed when referring to a negative or retarding acceleration. The instruments used for measuring acceleration, either positive or negative, are called accelerometers.

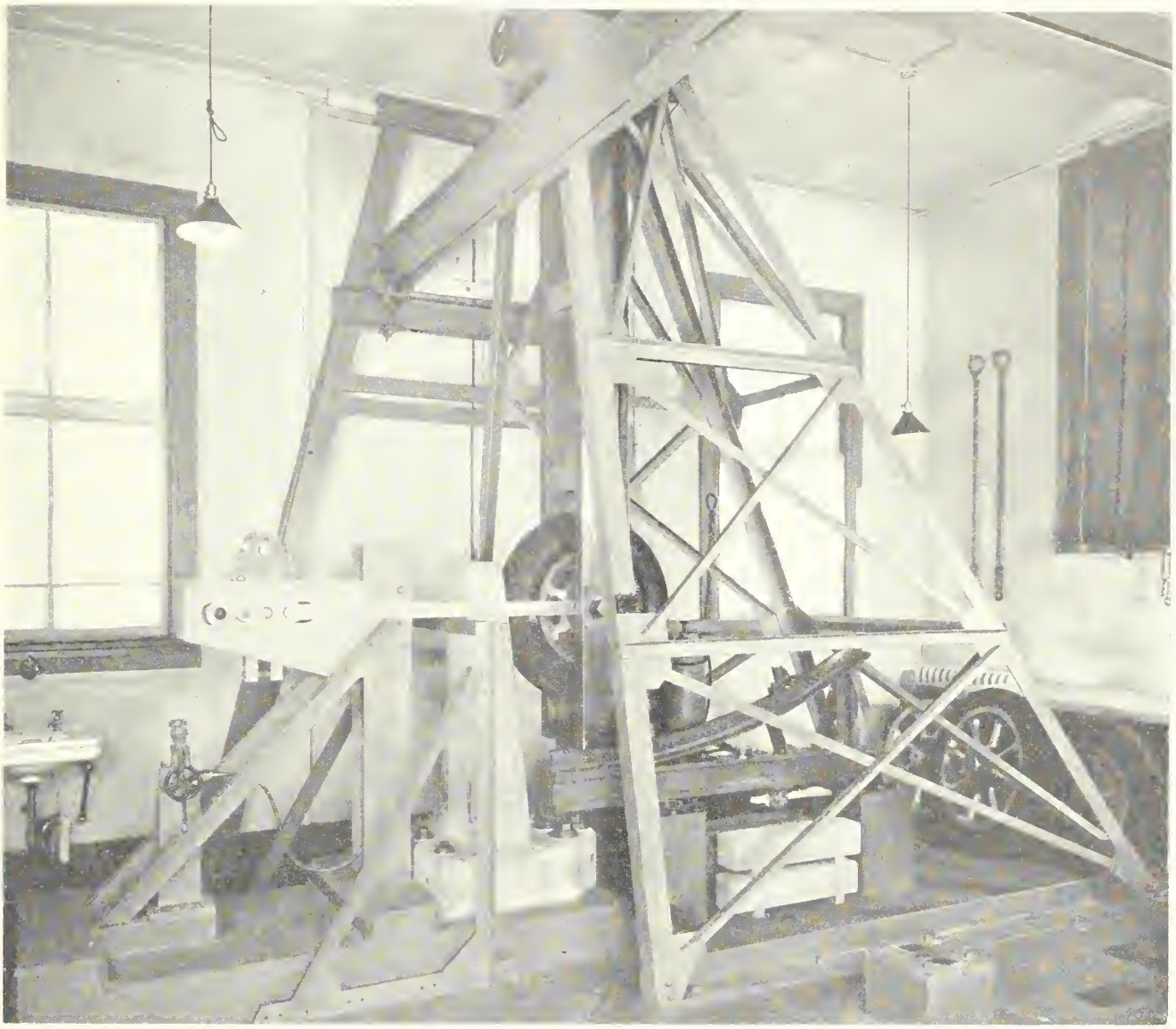


FIGURE 6.—GENERAL VIEW OF THE COMPLETE TESTING MACHINE.

should coincide. The pendulum length should be practicable yet sufficient to develop the desired forces with the mass available within a reasonable arc of swing. The maximum arc was arbitrarily set at 30°. Sufficient stiffness should be provided to minimize bending and torsional deformation in the pendulum proper.

The center of suspension should be so designed that the pendulum would swing true yet with the minimum of frictional damping. It should be rigidly supported yet adjustably mounted.

Acceleration of the pendulum mass at the center of percussion should be readily measurable.

Means should be provided for easily applying and accurately measuring sustained forces on the test specimens. Conversion from one method of force application to the other should be possible in a minimum of elapsed time.

Wheel and tire assemblies should be readily interchangeable, yet should be securely mounted in the pendulum. The range in tire diameters likely to be used would be 30 to 42 inches and the maximum sectional width of tire about 10 inches.

A positive pendulum release mechanism is necessary to permit exact duplication in the dynamic tests. An automatic rebound catch should be provided to permit single applications of dynamic force. Safeguards are necessary to prevent the accidental release of the pendulum.

The concrete cantilever beams should be conveniently placed and securely mounted in the testing apparatus. Access to all parts of the test specimen must be provided for strain and deflection measurements.

THE TESTING MACHINE DESCRIBED

The testing machine that was designed to meet the various requirements that have been described is shown in figure 6.

The pendulum element proper consists of two 10-inch aluminum alloy channel members arranged back to back and separated 11½ inches by four box-type stiffening diaphragms. The striking wheel is mounted at the center of percussion. Each of the four wheels is fitted with a special hub designed to receive steel centering cones through which passes a 2-inch diameter

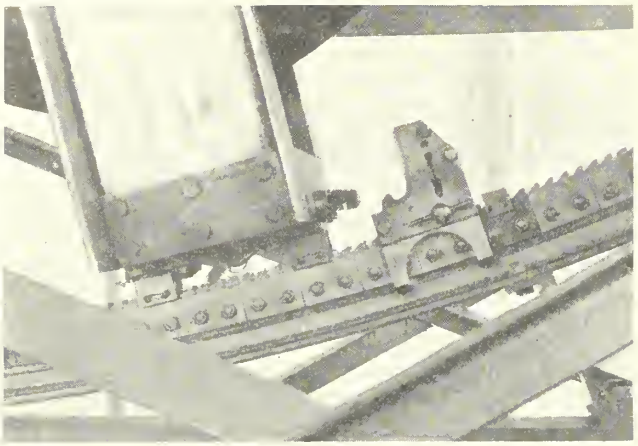


FIGURE 7.—THE PENDULUM RELEASE MECHANISM. NOTE THE REBOUND CATCH ENGAGED WITH THE RACK, UNDER THE CENTER OF THE PENDULUM.

steel shaft. Nuts on the ends of this shaft clamp the wheel assembly securely to the channel members.

At the upper end of the pendulum element is the axis of suspension or pivot. This consists of a steel shaft 1½ inches in diameter, which oscillates in two annular ball bearings.

The aluminum channel members are reinforced with steel plates at the points where the wheel and pivot shafts pass through them. One of these plates at each side of the wheel mounting is extended to serve as a support for one of the two accelerometers used in determining the impact forces. The length of the pendulum between the center of the pivot shaft and that of the striking wheel shaft is 100.00 inches.

The pivot bearings are carried in a heavy casting which is supported at the top of a pyramidal, structural steel frame or tower. The mounting permits adjustment of the pendulum to swing in a vertical plane properly oriented with respect to the test specimen and also permits a horizontal adjustment of the pivot in this plane of more than 6 inches. This traversing of the pivot is accomplished by the hand-wheel near the top of the tower (fig. 6) and the adjustment is made to compensate for differences in the radii of the several tires. After adjustment, the pivot casting is rigidly clamped to the tower by means of heavy bolts.

A rebound catch is mounted at the center of the lower end of the pendulum element. It consists of a pawl or detent mechanism which is automatically released during the impact. During rebound this pawl slides over the teeth of the large curved rack arranged along the arc of swing (see fig. 6) and engages at the end of swing, thus preventing a second or rebound blow from being struck. The large rack is independently supported on a heavy concrete pedestal and, since its position with respect to the axis of suspension must always be the same, it also is fitted with adjustments that allow this relation to be maintained whenever the pendulum pivot is moved.

In order to release the pendulum and allow it to swing through any desired angle (up to the 30° limit previously mentioned), a trip mechanism was designed that would free it suddenly, positively, and without imparting any tangential force. This mechanism, shown in figure 7, is adjustably mounted on the rack and can be locked at any desired point. A hook or catch on the back of the pendulum engages a similar element in the release device until freed by operation of the trigger lever. A pin lock prevents accidental operation of the release.

In order to make the wheel and tire assemblies dynamically interchangeable, the weights of these assemblies were adjusted to a common value by filling portions of the hollow steel wheels with lead. This additional weight was so distributed as to produce the least possible effect on the moment of inertia. The striking mass of the pendulum is thus the same regardless of the wheel and tire assembly used. The unavoidable variation in the moment of inertia of the wheel and tire assemblies produces a variation in the location of the center of percussion within the limits of 0.2 inch above to 0.3 inch below the center of the area of impact force application. The horizontal forces developed at the axis of suspension by this lack of complete coincidence are so small as to be without significance.

Each component part of the pendulum assembly when completed was carefully measured and weighed by two observers. From these data, the distance from the axis of suspension to the center of gravity of the part, the mass moment, and moment of inertia of the part about this axis, were computed. The observed weights, measured distances, and computed moments for all of the individual elements of the pendulum assembly are shown in table 2.

TABLE 2.—*Properties of the pendulum elements*

Element	Weight	Distance from center of gravity of element to axis of suspension	Moment of element about axis of suspension	Moment of inertia of element about axis of suspension
	Pounds	*Inches*	*Pound-inches*	*Pound-inches*[2]
Aluminum side channels	186.43	61.29	11,426	956,596
Pivot shaft, nuts, washers	15.40	.60	9	8
Pivot plates, bolts, etc	37.30	.13	5	397
Upper aluminum diaphragm, bolts, etc	15.12	8.00	121	1,179
Intermediate aluminum diaphragm, bolts, etc	14.94	42.00	627	26,563
Lower aluminum diaphragm, bolts, etc	15.03	76.00	1,142	87,023
Bottom cast-iron diaphragm, plates, screws	62.13	123.89	7,697	954,157
Wheel shaft plates, bolts, etc	31.60	100.00	3,160	316,429
Wheel shaft, cones, washers, nuts	28.22	100.00	2,822	282,235
Wheel and tire assembly	310.59	100.00	31,059	[1] 3,149,034
Accelerometers, bolts, etc	7.29	99.39	726	72,347
Accelerometer wires, binding posts	1.20	57.83	69	5,504
Rebound catch and trip assemblies	10.94	125.49	1,373	172,416
Summation—by first observer	736.19		60,227	6,023,888
Check—by second observer	736.24		60,226	6,023,307
Average values	736.22		60,226	6,023,598

[1] This is an average value for the 4 tire and wheel assemblies, which had values as follows: Balloon—3,136,683; high pressure—3,136,011; cushion—3,165,487; solid—3,157,954. This causes a variation in the location of the center of percussion of about −0.2 to +0.3 inch.

FORMULAS GIVEN FOR COMPUTING IMPACT FORCES

From the data in this table, the physical properties of the completed rigid pendulum were determined by means of the theoretical relations that were described earlier in the paper, as follows:

$\Sigma W = 736.22$ pounds, the total pendulum weight, acting at the center of gravity (point A, fig. 5).

$\Sigma M_0 = 60{,}226$ pound-inches, the summation of moments of all of the pendulum parts about the axis of suspension (point O, fig. 5).

$\Sigma I_0 = 6{,}023{,}598$ pound-inches,[2] the summation of moments of inertia of all of the pendulum parts about the axis of suspension.

k_0=90.453 inches, the radius of gyration of the composite pendulum about the axis of suspension.

$x=\frac{\Sigma M_0}{\Sigma W}$=81.804 inches, the distance of the center of gravity from the axis of suspension.

$l=\frac{k_0^2}{x}=\frac{\Sigma I_0}{\Sigma M_0}$=100.015 inches, the distance of the center of oscillation (or percussion) from the axis of suspension, also the length of the equivalent simple pendulum. (The designed length of l was 100.00 inches.)

$w=\frac{\Sigma M_0}{l}$=602.26 pounds, the pendulum weight effective at the center of percussion during impact.

g=32.155 feet per second per second, gravitational acceleration at the site of the pendulum.[7]

As previously stated the design of the pendulum is directly related to that of the test specimen because it must be capable of developing the forces required for the test. Mention has also been made of the fact that this force is dependent not only on the mass of the pendulum but on the acceleration or the time rate at which the velocity is reduced during impact, as well. In order to design successfully it was necessary to consider the complete interrelationship of the specimen, pendulum, and tire characteristics. The manner in which this was done will be described.

The concrete cantilever specimens are so placed that when the pendulum hangs freely at rest, the tire just touches the face of the specimen. The falling pendulum will therefore attain its maximum tangential velocity at the instant that the tire first makes contact with the specimen.

For the purpose of estimating probable acceleration maxima during impact, for use in the design, it was assumed that: (1) The combined cushioning of the tire and the specimen deflections will decelerate the pendulum in accordance with the laws of simple harmonic motion, (2) the influence of gravity on the deceleration of the pendulum is negligible, and (3) the center of percussion of the pendulum coincides with the center of the area of contact between the tire and the specimen. It is realized that none of these assumptions is rigorously true. However, each is sufficiently near the true condition to permit equating the maximum tangential velocity of the pendulum with the maximum linear velocity for simple harmonic motion to determine the probable maximum deceleration values for various tire types, angles of swing, and other conditions of test, with an accuracy that is sufficient for designing.

The physical relations involved during impact are:

d=the combined deflection of the tire and specimen or distance in which the pendulum is brought to zero velocity.

$v=\omega\theta l$, the maximum tangential velocity of the pendulum mass.

$A=\frac{v^2}{d}$, the maximum deceleration of the pendulum mass. (The velocity of the falling pendulum and that for the simple harmonic motion are assumed to be equal at the instant of contact.)

$F=\frac{wA}{g}$, the maximum force developed against the specimen (w being the effective weight of the pendulum).

$\frac{t}{2}=\pi\sqrt{\frac{d}{A}}$, the time duration of the contact between the tire and the face of the test specimen.

As already noted in connection with table 1, the influence of the angle of swing, as expressed by the term $\frac{\theta}{\sin\theta}$, or its reciprocal, is negligible for small angles and not important for angles of moderate magnitude [8] except in calculations where great precision is desired. In computing the probable values of the maximum accelerations the value of this term was assumed to be unity, permitting desirable simplification in a number of instances.

VIBRATION AND DEFLECTION OF SUPPORTING FRAME INSIGNIFICANT

Because of the fact that during the period of tire and specimen deflection the pendulum mass is not moving exactly horizontally but in a rising arc, the effective force applied to the specimen is slightly less than the apparent force. This difference amounts to about 6 pounds per inch of tire deflection and applies to both the sustained and the impact forces.

By the method outlined, the relations between the angle of swing, tire and beam deflection, and maximum impact force, were calculated and these are shown graphically in figure 8. To this chart have been added the curves from figure 4 that show the static load deflection characteristics of the tires, and the curves from figure 3 that show the theoretical bending stress and deflection characteristics of the cantilever test specimens. Thus figure 8 presents a composite graph showing the general relationships that exist between the various factors that are involved in a test. This graph has proved to be quite useful in connection with actual testing operations as a guide for selecting the conditions necessary to produce certain desired results.

As a check on the correctness of the calculated values for mass and center of percussion, the location of the center of gravity and the period of free swing were determined empirically. This was done after the pendulum was substantially completed. At the time of the determination the computed distance between the axis of suspension and the center of gravity was 81.36 inches. The mean value of several experimental balancings gave a value of 81.33 inches, a difference of less than 0.04 percent. Subsequent slight changes in the pendulum changed the value to that given earlier in the paper (81.804 inches).

The period of swing was determined experimentally by means of stop-watch measurements of the time required for 50 cycles. The damping caused by air and pivot friction as reflected in the reduction in the length of arc for successive swings (measured horizontally) was observed also. The data obtained from each of these observations for series of swings of small amplitude are shown in figure 9. It is noted that the measured period varies with the angle of swing and that it is within about 0.005 second of the theoretical value. No attempt was made at a precise determination. The damping also varies with the angle of swing although within the range covered by the experiments the loss per cycle is quite small.

By measuring the maximum deceleration that occurs at the center of mass concentration when the mass of

[7] From the Smithsonian Physical Tables, 8th edition, p. 564.

[8] For an angle of 10° the error caused by assuming the value of this term to be unity amounts to less than ¼ percent.

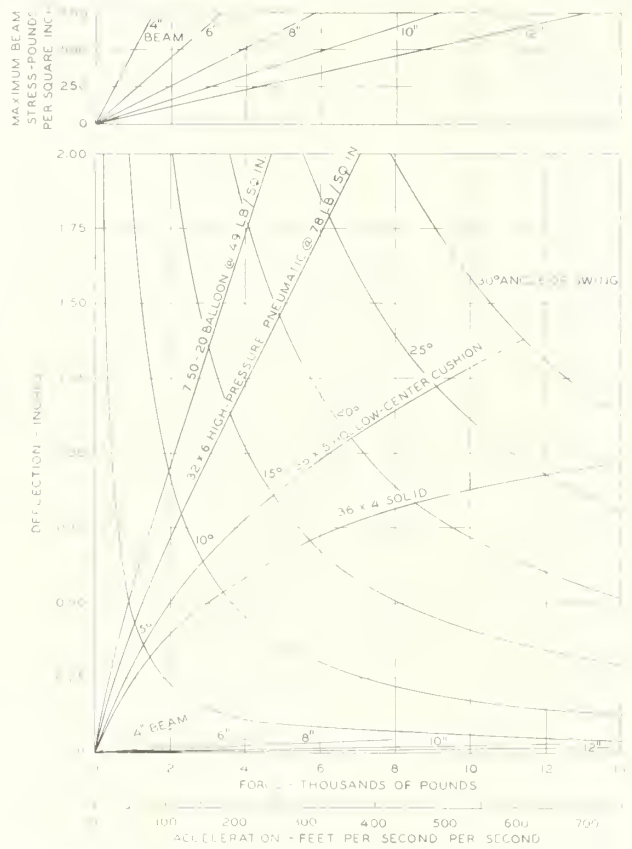


FIGURE 8.—COMPOSITE GRAPH SHOWING INTERRELATED CHARACTERISTICS OF TEST SPECIMENS, TIRES, AND PENDULUM.

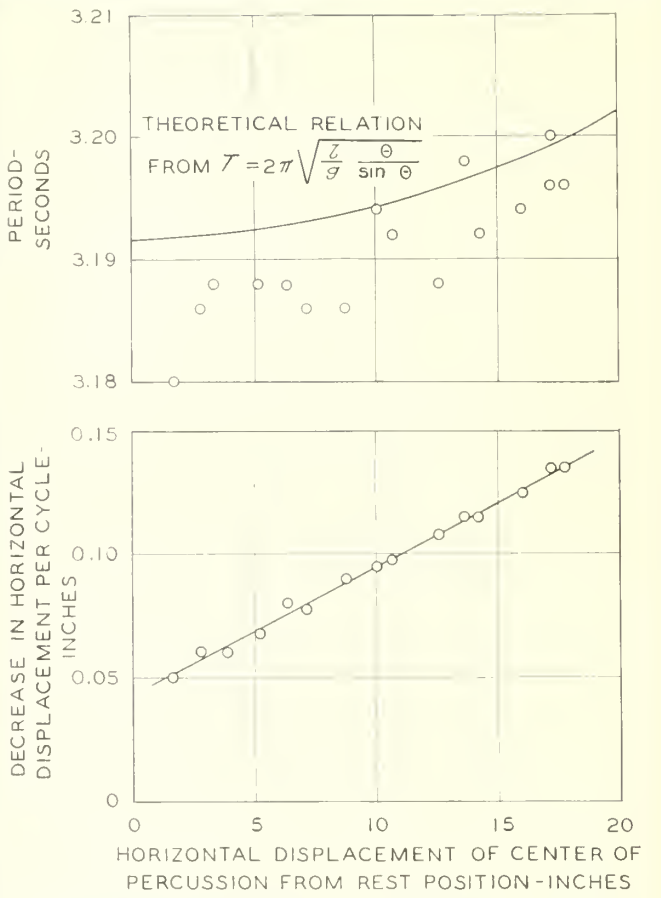

THEORETICAL RELATION
FROM $T = 2\pi\sqrt{\frac{l}{g}\frac{\theta}{\sin\theta}}$
PERIOD - SECONDS
DECREASE IN HORIZONTAL DISPLACEMENT PER CYCLE - INCHES
HORIZONTAL DISPLACEMENT OF CENTER OF PERCUSSION FROM REST POSITION - INCHES

FIGURE 9.—FREE SWINGING NATURAL PERIOD AND DAMPING OF PENDULUM FOR VARIOUS DISPLACEMENT DISTANCES.

the pendulum is brought to rest during the deflection of the tire and the specimen, the maximum dynamic force can be calculated. For measuring the maximum acceleration values, accelerometers were mounted on the pendulum in the manner shown in figure 10. The accelerometers are mounted one on each side opposite the center of impact. The instruments are of the single-element, electrically indicating, contact type described fully in earlier publications.[9]

The design and construction of the pendulum are such that there is essential coincidence of the center of percussion with the center of the area of dynamic-force application. It was mentioned earlier that when this condition obtains no force reaction is created at the center of suspension during impact. Since there is a negligible change in the forces acting at the center of suspension, vibration and deflection in the supporting frame are reduced to a minimum.

APPARATUS FOR APPLYING SUSTAINED LOADS DESCRIBED

Earlier in the paper mention was made of the development of sustained forces against the specimen by a pull exerted on the wheel shaft of the pendulum. The general appearance of the apparatus for this purpose can be seen at the left of the specimen in figure 6. Briefly, it consists of a suitably arranged hydraulic jack and pump for developing the force and an elastic system for measuring its magnitude.

A heavily reinforced concrete platform serves as a base for the sustained-load apparatus and as an anchorage for the test specimens. At a position in the rear of the specimen, a post consisting of an 8-inch H-section is erected and rigidly braced to structural steel members encased in the concrete base. This post supports the sustained load apparatus and also carries a buffer to take the test specimen should it fail unexpectedly. A hydraulic jack of 5-inch diameter is secured to the post in such a manner that its axis lies in the plane of the pendulum and at the height of the wheel shaft. The line of travel of the jack ram is horizontal and its direction of motion is away from the test specimen.

The H-beam post supports also a pair of heavy wing plates, one on either side, which carry the load-measuring equipment. Horizontal slots or openings cut in these plates provide level runways for a carriage that rolls on annular-ball-bearing wheels. A sturdy transverse reaction pin or shaft receives the thrust of the jack through the load-measuring device, also supported by the carriage. Connection between the reaction pin and the wheel shaft of the pendulum is effected by a pair of steel tie bars, one of which can be seen in figure 6. A series of eyes at one end of each of these bars provide engagement at any one of four positions for the reaction pin while at the other end of the bars quickly detachable latches furnish a simple means for attachment to the wheel shaft through the conical blocks that center the wheel hub. The connection between the tie bars and the reaction pin is made through adjustable eccentric bushings which permit exact parallelism to be obtained between the reaction pin and the wheel shaft of the pendulum.

The load-measuring device consists of a pair of opposed, simply supported, vertically mounted steel

[9] See Calibration of Accelerometers for Use in Motor Truck Impact Tests, by J. A. Buchanan and G. P. St. Clair, PUBLIC ROADS, vol. 11, no. 5, July 1930, also, Impact Reactions Developed by a Modern Motor Bus, by J. A. Buchanan, PUBLIC ROADS, vol. 12, No. 2, April 1931.

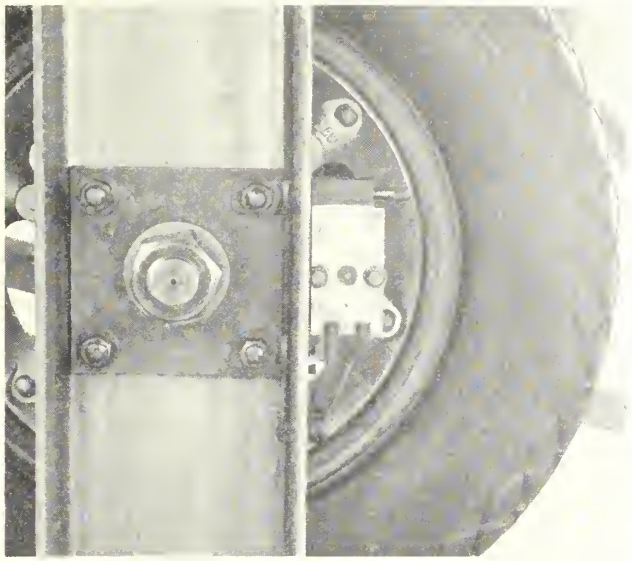


FIGURE 10.—CONTACT ACCELEROMETER MOUNTED ON CHANNEL MEMBER DIRECTLY OPPOSITE CENTER OF PERCUSSION.

beams of heat-treated alloy steel, whose combined deflections, under a centrally-applied load is measured with a dial micrometer as an index of the load or force. At each end of the inner faces of the beams there is a common reaction bearing in the form of a ground pin or roller in suitable seating blocks. At the upper end of the beams as they are mounted, the bearing pin is extended in both directions and these extended ends provide centers for a pair of annular ball bearings. These bearings roll on short horizontal tracks provided in the carriage which bears the reaction pin. This suspension permits the pair of beams to hang freely and vertically between the jack and the reaction pin. The details of the suspension are shown in figure 11.

The head of the hydraulic jack is fitted with a bronze face 3.8 inches square for bearing on one of the beams. The reaction pin which bears on the opposed beam is of heat-treated alloy steel ground to a diameter of 2 inches. Each beam is of alloy steel, heat treated and ground to a true rectangular prism 3.8 inches wide and 1.8 inches deep. The distance between reactions (or span) is 20 inches.

The combined deflection of the two beams is measured at the midpoint with a dial micrometer which reads directly in ten-thousandths of an inch. One division on the dial of the micrometer corresponds to an applied load increment of about 15 pounds, this being determined by calibration in a testing machine. The safe load-carrying capacity of this particular pair of beams is ample for all requirements and their elastic behavior has been constant.

The cantilever test specimens are securely mounted on the reinforced concrete platform in the position shown in figure 6. Horizontal sliding is prevented by friction on the base and by the reaction developed at the low transverse abutment that is an integral part of the platform. Moments tending to overturn the specimen are resisted by 10 large anchor bolts that pass through the base of the specimen. Seven of these bolts, each 1½ inches in diameter, are on the side nearest the pendulum, while three bolts, each 1 inch in diameter, are on the side nearest the vertical post. The stress in these bolts is relatively small, their design being such that, for the conditions assumed,

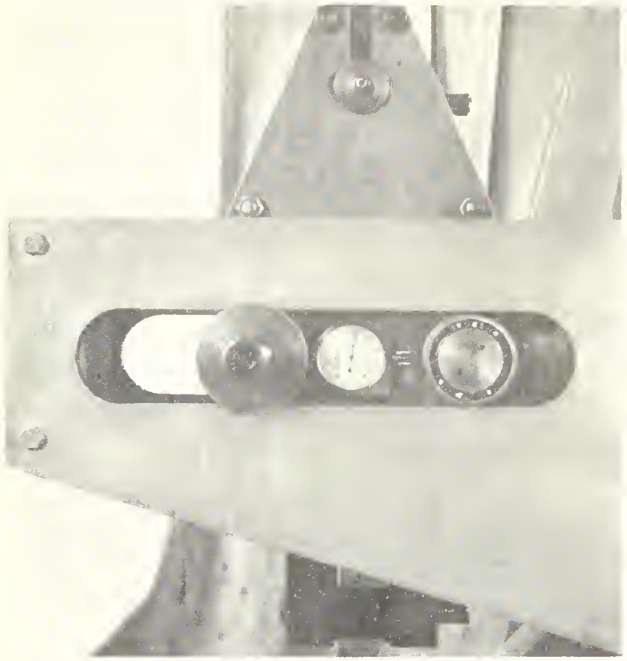


FIGURE 11.—DETAILS OF THE SUSTAINED LOAD DEVICE. SEEN THROUGH THE OPENING OF THE WING PLATE FROM RIGHT TO LEFT ARE RAM OF THE JACK, BALL-BEARING CARRIAGE WHEEL, DIAL MICROMETER BETWEEN THE PAIR OF BEAMS, AND THE END OF THE REACTION PIN. ABOVE IS THE CENTER OF SUSPENSION FOR THE LOAD-MEASURING BEAMS.

their maximum elongation would not cause an increase in the measured deflection of the specimen of more than about 5 percent. In the great majority of the tests the effect would be considerably less than this.

STRAINS AND DEFLECTIONS OF CONCRETE BEAMS MEASURED

An independently supported trolley and a geared hoist afford means for swinging the cantilever test beams into position on the platform anchorage. The beams are lifted by a sling attached to the ends of two steel bars which are passed through longitudinal holes in the specimen base (see cover illustration). The specimens are first placed on temporary wedges and carefully adjusted for elevation, alinement, perpendicularity, and orientation. The anchor bolts are then inserted but not tightened. The entire space between the platform and the base of the specimen (which rests on the wedge supports) and the space between the specimen base and the rear abutment is next filled with portland cement mortar placed with special tools and thoroughly compacted. After this bedding material has set sufficiently to bear the weight of the specimen without flowing, the wedges are removed. The anchor bolts are tightened after the mortar under the base has set thoroughly. The bolt heads bear on the top of the base through large cast iron washers set in mortar.

The elastic behavior of the test specimen is studied by comparing strains in the compression and tension faces of the cantilever and deflections under both sustained and suddenly applied forces of corresponding magnitudes.

The strains are measured with the recording strain gage developed by the Bureau and described in detail elsewhere.[10] In addition to the static calibration described in this reference, the behavior of the gages

[10] An Improved Recording Strain Gage, by L. W. Teller, PUBLIC ROADS, vol. 14, No. 10, December 1933.

FIGURE 12.—STRAIN GAGES ON THE COMPRESSION FACE OF THE TEST SPECIMEN. A CORRESPONDING PAIR ON THE OPPOSITE FACE MEASURES TENSILE STRAINS.

under suddenly applied strains has been investigated in several ways and for the rates of force development that are encountered in motor-vehicle impact there appears to be no significant difference between the behavior under displacements developed slowly and those developed suddenly.

Strain gages are placed at corresponding positions near the bottom of the compression and tension faces of the cantilever beam, but sufficiently above the base to avoid significant modification of stress due to the influence of the base. Strain gages located on the compression face are shown in figure 12.

The deflection of the specimen is usually measured as a horizontal displacement of a point on the compression face opposite the center of the area of force application. The apparatus consists of a light stylus arm extending from a plug set in the concrete of the specimen and arranged to record the displacement directly on a smoked glass plate that is held in guides on an independent structural steel frame. This frame can be seen astride the concrete platform, just in back

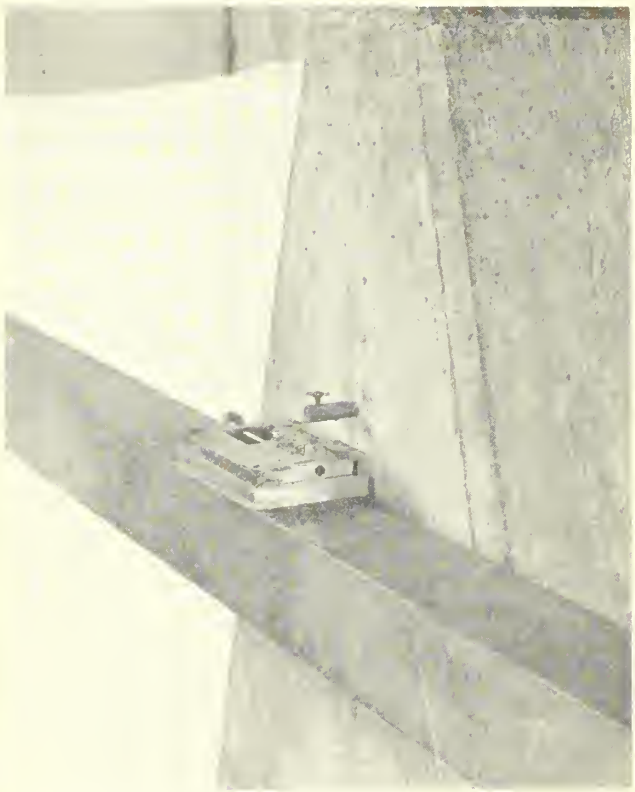


FIGURE 13.—BEAM DEFLECTION IS RECORDED ON A SMOKE-COATED GLASS PLATE BY MEANS OF A STYLUS.

of the specimen in figure 6, and the device itself is shown in figure 13.

The machine is erected in a building having a very heavy floor. This floor consists of a 12-inch reinforced concrete floor slab cast with heavy integral girders and supported on concrete walls at the edges and short columns at the third points in both directions. While no provision is made for maintaining a constant temperature in this laboratory, an effort is made to avoid widely fluctuating temperatures, particularly sudden changes.

The first group of specimens fabricated contained 26 units, the majority of which were of intermediate thickness. Of this group a considerable number have been tested and many thousands of load applications have been made. This experience has demonstrated that the testing equipment is satisfactorily producing the results for which it was designed.

DETERMINATION OF VARIATION IN UNIT PRESSURE OVER THE CONTACT AREA OF TIRES[a]

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Senior Engineer of Tests, and JAMES A. BUCHANAN, Associate Engineer of Tests

ONE of the lesser but persistently recurring questions that arise in considering the effects of wheel loads on pavement surfaces is that of the variation in pressure intensity over the area of contact between the tire and the pavement.

In the early attempts at stress analysis in pavement slabs it was assumed that the load was applied either at a point or along a line whose length equaled the width of the tire. Later, as pneumatic tires came into more widespread use, it was recognized that these assumptions were far from true. When Westergaard presented his theory of stresses in pavement slabs before the Highway Research Board in 1925,[1] he used the assumption that wheel loads are applied to pavement slabs through a circular area of contact over which the unit pressure is uniform.

This assumption had the advantage of simplicity; and, for purpose of analysis, it seemed to be not unreasonable, particularly as applied to pneumatic tires. It was known, of course, that the contact area was not actually circular, nor could the unit pressure over it be strictly uniform because of inherent characteristics in the form and construction of the tire. However, at that time there was little or no information that could be used as a basis for a better assumption. The shapes of the areas of contact have since become more generally known, but the manner in which unit pressure varies over such areas has remained relatively unknown.[2]

The reason for this scantiness of available data probably lies in the lack of a simple and satisfactory means for making reliable determinations.

In an effort to obtain additional information on this subject, the Bureau of Public Roads has recently developed the following method for determining these unit pressures—a method which requires but little addition to readily available equipment and which appears to give reasonably good results.

TEST EQUIPMENT AND PROCEDURE DESCRIBED

The tire is mounted in a universal testing machine in the manner usually employed in load-deflection tests. A smoothly ground steel plate of adequate size is placed to receive the reaction. Between the tire and the reaction plate is placed a $\frac{5}{16}$-inch square bar of polished brass, arranged longitudinally along the area of contact. On either side of this bar are set steel filler plates of the same thickness as the bar and of sufficient size so that the bar and plates cover the entire area of contact. A windlass and spring dynamometer provide a means for applying and measuring the force required to overcome initial friction in sliding the bar longitudinally while load is applied to the tire. No lubricant is used, but a sheet of steel shim stock 0.0018 inch thick is placed between the tire and the friction bar to provide a uniform contact surface for sliding. The manner in which the equipment is set up in the testing machine is shown in figure 1 and the details of the friction bar, filler plates, and shim stock cover, are shown in figure 2.

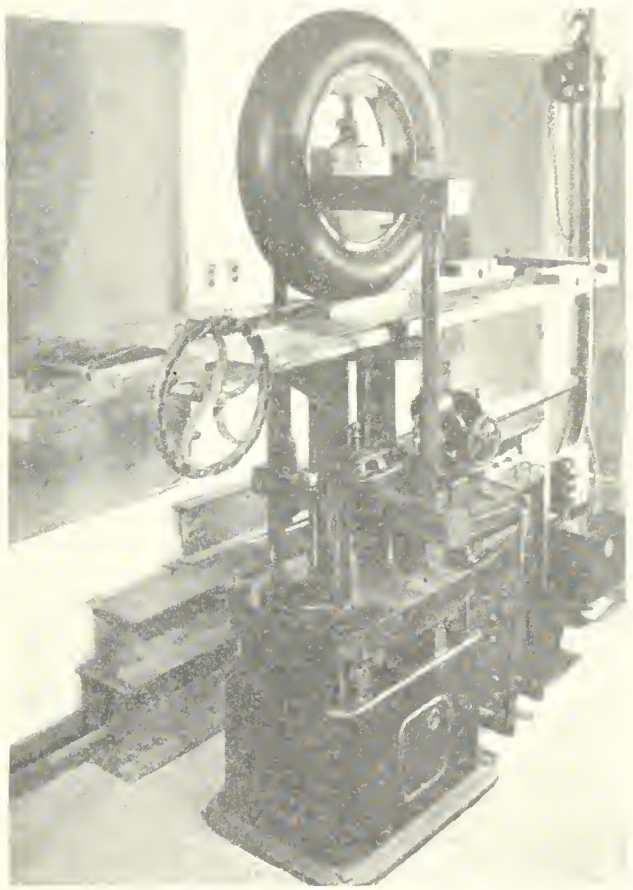


FIGURE 1.—TIRE MOUNTED IN A UNIVERSAL TESTING MACHINE EQUIPPED TO MAKE UNIT PRESSURE DETERMINATIONS.

In making the test the procedure is to place the friction bar successively in parallel positions $\frac{5}{16}$ inch apart until the entire area of contact has been covered. In each position the force required to move the bar longitudinally in each direction is measured and the two determinations are averaged. The test load is

[a] Paper presented at the Seventeenth Annual Meeting of the Highway Research Board, Washington, D. C., 1937.

[1] Computation of Stresses in Concrete Roads, by H. M. Westergaard. Proceedings, Fifth Annual Meeting, Highway Research Board, 1925.

[2] Glimpses of Balloon Tire Progress, by B. J. Lemon. Journal of the Society of Automotive Engineers, February 1925.

Distribution of Wheel Loads Through Various Ruhber Tires, by Samuel Eckels Proceedings, Eighth Annual Meeting, Highway Research Board, 1928.

Tests to Determine the Behavior of Tires on a Smooth Surfaee (Onderzoekingen Omtrent het Gedrag van Autobanden op een Effen Weg) Engineering Thesis, Technische Hoogeschool, Delft, hy Hendrik Misset, January 20, 1932. (Particularly chapter 3—The distribution of pressure in the contaet area between the tire and the road.)

Ground Contact Area of Tires Varies Directly with Deflection, hy P. M. Heldt, Automotive Industries, July 23, 1932.

Semi-Pneumatie Tires and Speeific Pressures (Semipneumatici e Pressioni Specifiche) by Raffaele Ariano. Le Strada (Italian), November 1933.

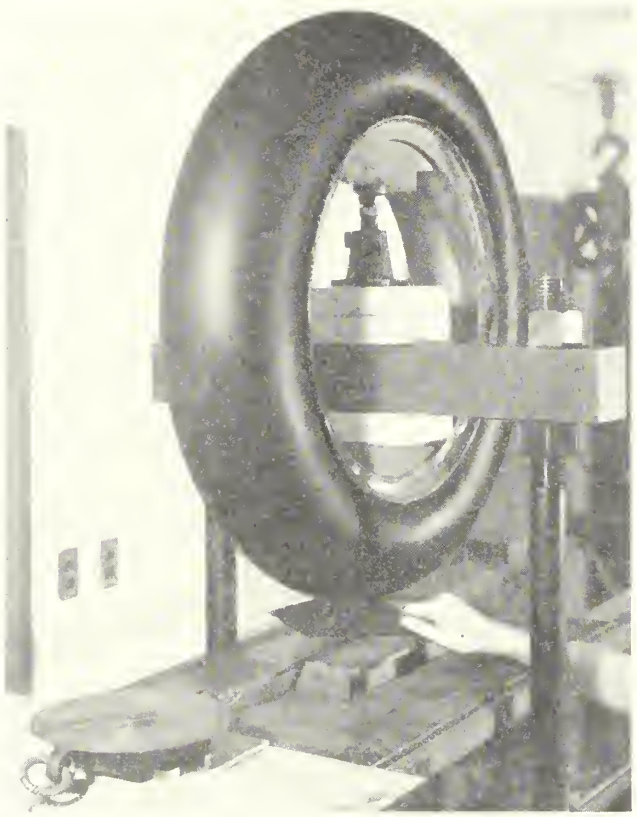


FIGURE 2.—DETAILS OF THE APPARATUS. THE FRICTION BAR LIES BETWEEN FILLER PLATES OF EQUAL THICKNESS AND IS ATTACHED TO THE SPRING DYNAMOMETER. THE FRICTION SURFACES, WHEN THE TIRE IS UNDER LOAD, ARE BETWEEN THE BOTTOM OF THE FRICTION BAR AND THE LOWER PLATE OF GROUND STEEL, AND BETWEEN THE TOP OF THE FRICTION BAR AND THE STEEL SHIM PLACED UNDER THE TIRE.

assumed to be distributed to the respective elemental strips or sections thus obtained, in proportion to the ratio of the average frictional force for each section to the summation of the average frictional forces for all sections. In effect, this amounts to a first differentiation of the test load and is diagrammatically represented in figure 3, A.

To obtain the distribution along each of the longitudinal sections $\frac{5}{16}$ inch wide, the differences between the frictional resistances for successive increments of contact length on the sliding bar are determined, the remaining portion of the longitudinal strip being filled with a separate and stationary square bar. A convenient length increment is 1 inch and, as in the first differentiation, the average frictional resistance for both directions of movement is obtained.

The total load on each elemental strip is apportioned along its length according to the force differences for its respective subsections. By the double differentiation thus accomplished, the total load over the entire contact area is broken down into a great many small loads on elemental areas each $\frac{5}{16}$ inch wide and 1 inch long, except in the boundary areas where irregular shapes are unavoidable. These elemental areas are shown in figure 3, B.

Assuming the unit pressure at the center of gravity of each of the elemental areas to be the average intensity of pressure over that area, the unit values are computed and plotted in their respective positions on an outline of the actual gross contact area for the load being used in the test. The construction of the

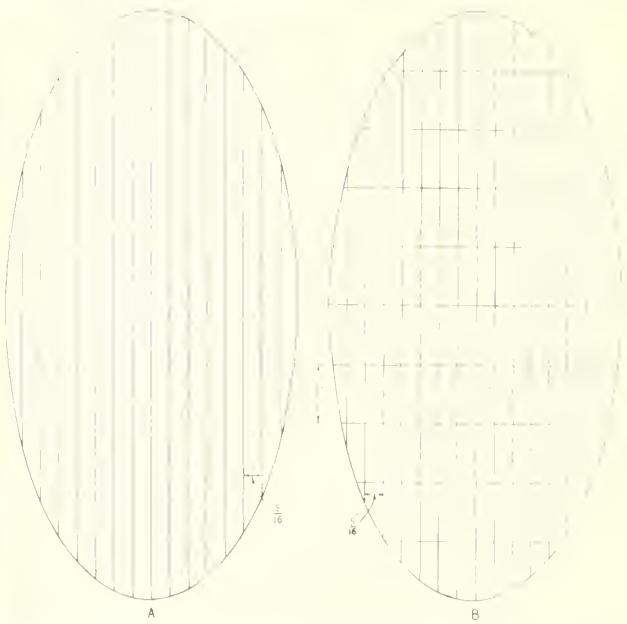


FIGURE 3.—DIAGRAM SHOWING HOW CONTACT AREAS WERE DIVIDED (A) INTO SECTIONS $\frac{5}{16}$ INCH WIDE, AND (B) INTO AREAS $\frac{5}{16}$ INCH BY 1 INCH, FOR THE FIRST AND SECOND DIFFERENTIATIONS OF THE TEST LOAD, RESPECTIVELY.

FIGURE 4.—THE SOLID AND BALLOON TIRES USED IN MAKING UNIT PRESSURE DETERMINATIONS.

lines or contours of equal unit pressure within this area is then a simple process.

Two tires have been tested by this method—one a 36- by 6-inch solid truck tire, the other a 36- by 8-inch airplane pneumatic tire. Each was tested under a 4,000-pound load, corresponding approximately to the normal rated capacity. These two tires were selected for the initial tests for two reasons. First, they both had smooth treads, thus eliminating any question as to the effect of tread design and giving more basic data on the distribution of unit pressure. Second, they represent about as wide a range in cushioning quality as will be commonly found in tires of their capacity on the highway today.

The solid tire had become hardened somewhat with age, although it had been but slightly used. The pneumatic tire was mounted on a 36- by 8-inch truck tire rim and inflated to 60 pounds per square inch. They are shown in figure 4.

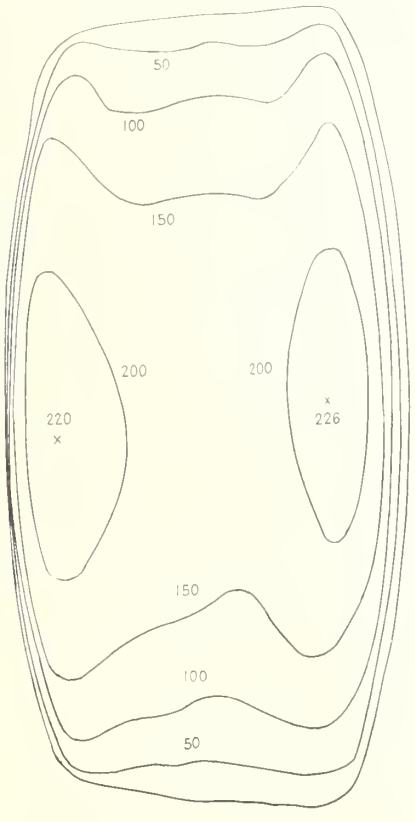
MAXIMUM INTENSITY OF PRESSURE UNDER SOLID TIRE THREE TIMES THAT UNDER PNEUMATIC TIRE

The unit pressure distributions over the contact areas of the two tires, obtained in the manner described, are shown in figures 5 and 6.

The intensity distribution data for the pneumatic tire show that over a considerable portion of the contact area the intensity of pressure is essentially uniform and not greatly different from the inflation pressure of the tire. For the solid tire, the zone of approximately constant unit pressure covers a much smaller percentage of the total area and the intensity within the zone is much higher than for the pneumatic tire. These relations are as might be expected from a consideration of the nature of the cushioning media involved.

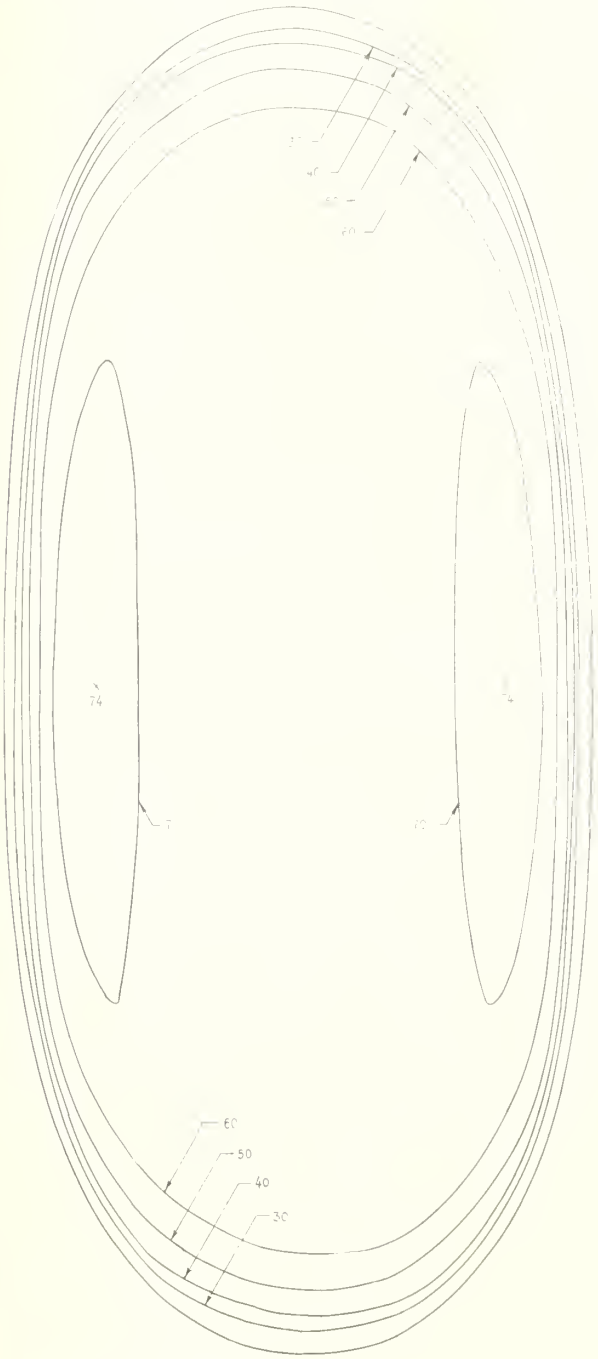
However, when consideration is given to the extreme difference in type represented by these two tires, the similarity between their unit pressure patterns is rather remarkable. The variations in unit pressure have the same general characteristics and the maximum values occur in two transversely symmetrical areas rather than at the geometric centers of the contact areas. The solid volumes defined by the contours of figures 5 and 6 represent the total loads supported by the tires. As a check on the work, these volumes were computed and converted into equivalent load. The results of these integrations are indicated on the figures.

The dimensions of the two contact areas and corresponding unit pressure values are given in table 1. In addition to the maximum measured intensities, values are also given for the corresponding uniformly loaded circular areas previously mentioned and for assumed ellipsoidal distributions in which the intensities of



TIRE SIZE 36 BY 6 INCHES
APPLIED LOAD____4,000 POUNDS
INTEGRATED LOAD___3,992 POUNDS

FIGURE 5.—UNIT PRESSURE DISTRIBUTION OVER THE AREA OF CONTACT FOR A REGULAR SOLID TIRE. FIGURES SHOW PRESSURE IN POUNDS PER SQUARE INCH.



TIRE SIZE 36 BY 8 INCHES
INFLATION PRESSURE 60 POUNDS PER SQUARE INCH
APPLIED LOAD____4,000 POUNDS
INTEGRATED LOAD___3,914 POUNDS

FIGURE 6.—UNIT PRESSURE DISTRIBUTION OVER THE AREA OF CONTACT FOR A LOW-PRESSURE PNEUMATIC TIRE. FIGURES SHOW PRESSURE IN POUNDS PER SQUARE INCH.

FIGURE 7.—COMPARISON OF ACTUAL UNIT PRESSURE DISTRIBUTION WITH THAT FOR A UNIFORMLY LOADED CIRCULAR AREA.

pressure are assumed to vary as the ordinates to semi-ellipsoidal surfaces placed over the areas of contact.

TABLE 1.—*Contact dimensions and pressure intensities for a load of 4,000 pounds on two types of tire equipment*

	Smooth-tread solid tire [1]	Smooth-tread pneumatic tire [2]
Contact area.....sq. in.	27.3	64.7
Contact length.....in.	7.8	13.3
Contact width.....do	4.0	5.8
Diameter of equivalent circular area.....do	5.82	9.05
Diameters of equivalent elliptical area.....do	8.32—4.16	12.76—6.38
Average unit pressure over contact area.....lb./sq. in.	146	62
Maximum measured unit pressure.....do	226	74
Maximum unit pressure for ellipsoidal distribution..do	220	93

[1] 36 by 6 inches.
[2] 36 by 8 inches, inflated to 60 pounds per square inch.

In figures 7 and 8 the actual transverse and longitudinal distributions are compared with those obtained by the uniformly loaded circular area method and by the suggested ellipsoidal distribution method. For the circular distribution, the area of the circle is equal to the actual area of contact; and the intensity, being the average pressure, does not vary. For the ellipsoidal distribution the area of the ellipse is equal to the actual area of contact; the diameters of the ellipse are in the ratio of 2 : 1; and the intensity varies over the area of contact in the manner described above. While it is obvious that neither of these conventionalized representations is an exact definition of actual conditions, it is evident from a comparison of figures 7 and 8 that the ellipsoidal distribution conforms much more closely to the actual than does the uniformly loaded circular area. This is also evident in the actual contour patterns of figures 5 and 6 and in the numerical data of table 1.

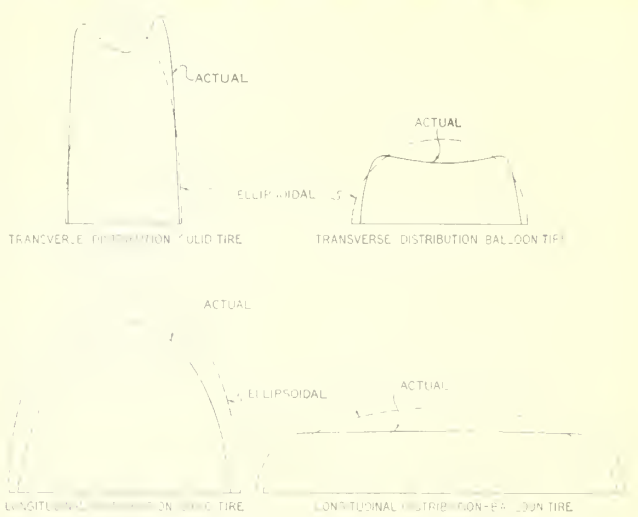


FIGURE 8.—COMPARISON OF ACTUAL UNIT PRESSURE DISTRIBUTION WITH ELLIPSOIDAL PRESSURE DISTRIBUTION.

The 2:1 ratio between the diameters of the ellipse is a simplification which was selected after a review of tire impressions obtained in nearly 400 tests of tires, tests that involved a wide variety of types, sizes, loads, and degrees of cushioning. This study showed an average ratio of contact length to contact width of approximately 2:1.[3]

The assumption that a vehicle tire transmits its load to the pavement through a uniformly loaded circular area is a generalization that may be of sufficient accuracy for most analytical purposes. The ellipsoidal distribution, however, is also capable of mathematical expression and, from the data obtained in these tests, appears to express more accurately the true condition. Whether or not differences in the manner of pressure distribution over the area of contact would lead to significant differences in pavement stress is a matter which may be studied either analytically or by physical tests. If such a study should reveal that variation in intensity of pressure is a matter of sufficient importance to be considered, then the ellipsoidal representation suggested by these tests offers a conveniently expressed generalization that reasonably approaches actual conditions of unit pressure distribution.

[3] For solid tires the average ratio was 1.59; for cushion tires, 1.95; for high-pressure pneumatic tires, 2.22; and for balloon tires, 2.12.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF NOVEMBER 30, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 583,100	$ 281,550	19.6	$ 2,470,361	$ 1,235,180	127.5	$ 2,838,620	$ 1,419,305	129.5	$ 4,911,145
Arizona	1,112,079	813,048	47.7	1,594,385	1,134,923	72.5	223,721	160,767	10.3	1,208,220
Arkansas	1,983,112	1,981,278	120.8	2,188,072	2,183,586	148.9	23,505	23,177		2,210,878
California	5,498,435	2,946,065	113.8	5,991,087	3,221,670	112.7	2,992,449	1,581,385	55.4	1,166,154
Colorado	2,579,505	1,440,469	101.6	1,106,257	608,843	34.2	1,200,474	651,578	43.4	1,680,199
Connecticut	783,198	389,196	9.3	231,520	108,123	.8				1,513,589
Delaware	231,150	115,530	5.8	305,782	152,847	15.2	219,285	107,624	13.3	1,117,011
Florida	301,250	149,953	8.2	2,716,172	1,358,086	65.2	508,170	254,085	7.8	2,655,059
Georgia	1,827,545	906,734	108.9	4,133,172	2,066,581	179.8	3,719,426	1,859,713	159.9	3,838,348
Idaho	1,939,457	1,157,184	162.3	1,152,613	688,471	79.6	324,130	192,020	15.4	934,782
Illinois	8,049,929	3,966,266	216.3	6,435,061	3,213,423	200.8	4,328,700	2,051,850	86.3	1,742,718
Indiana	4,201,035	2,094,684	97.5	4,143,915	2,071,841	120.7	1,561,323	761,349	35.9	1,721,973
Iowa	6,363,111	2,885,898	206.5	4,319,091	1,918,113	123.4	2,313,087	1,074,650	68.2	208,627
Kansas	2,871,112	1,407,608	164.4	3,397,445	1,698,621	108.0	2,028,609	1,013,032	103.1	3,213,482
Kentucky	1,358,902	629,451	40.5	3,598,362	1,799,181	105.6	1,697,590	848,795	82.8	2,405,857
Louisiana	382,218	186,969	9.3	6,555,335	1,430,435	45.2	5,293,838	836,455	12.5	1,729,722
Maine	1,898,980	949,490	52.6	1,831,416	915,708	41.7	444,429	222,210	10.0	191,803
Maryland	184,006	92,003	2.1	2,371,418	1,183,926	37.0	212,000	106,000	3.2	1,712,879
Massachusetts	1,405,641	702,820	9.6	4,834,281	2,417,139	13.9	107,346	53,673		1,914,700
Michigan	5,373,810	2,666,746	142.4	5,765,110	2,883,055	140.2	2,122,309	986,329	30.4	185,756
Minnesota	5,277,952	2,627,429	242.7	3,116,681	1,411,211	133.2	2,074,604	1,037,302	83.8	1,330,548
Mississippi	1,242,210	621,100	59.3	4,464,490	2,231,430	206.0	2,339,440	1,209,020	94.9	2,571,626
Missouri	6,061,226	3,007,848	353.2	5,848,769	2,806,918	182.9	3,802,495	1,576,761	88.0	1,421,548
Montana	3,721,299	2,093,228	277.7	1,648,307	889,815	60.1	1,158,967	651,907	67.9	1,850,663
Nebraska	2,098,156	1,049,078	205.5	4,602,599	2,289,278	440.4	2,576,516	657,149	79.6	2,136,261
Nevada	1,600,322	1,376,084	83.3	1,240,211	1,072,530	62.1	194,286	168,479	27.0	752,301
New Hampshire	217,825	107,616	3.3	495,198	244,730	9.8	196,021	96,847	2.9	985,626
New Jersey	1,674,609	760,009	20.2	841,670	420,515	2.0	1,033,280	515,805	10.7	1,784,824
New Mexico	2,400,300	1,473,364	191.6	2,827,723	1,824,005	137.3	114,353	69,739	1.8	280,256
New York	9,785,560	4,596,809	185.1	16,454,138	7,867,325	258.0	2,360,540	1,178,770	31.0	398,519
North Carolina	2,329,608	1,161,503	180.5	5,870,974	2,764,937	284.5	1,491,140	709,000	61.5	2,469,315
North Dakota	1,012,110	1,012,110	186.5	1,192,330	1,172,320	77.4	294,572	294,572	36.4	3,243,231
Ohio	2,426,602	1,139,650	30.1	9,995,419	4,966,354	112.7	2,077,260	1,038,630	25.6	4,945,536
Oklahoma	2,507,319	1,315,152	103.1	2,655,498	1,373,237	121.7	1,510,965	794,868	66.3	3,194,210
Oregon	3,380,430	1,994,181	122.8	1,536,520	937,055	54.1	842,035	475,028	44.4	877,471
Pennsylvania	7,979,161	3,970,260	121.5	8,236,069	4,098,185	105.7	5,180,236	2,580,379	83.3	2,226,381
Rhode Island	551,258	266,814	5.7	1,350,904	675,452	15.4	8,930	4,465		757,990
South Carolina	2,304,729	965,203	188.8	4,792,449	1,991,505	211.5	1,405,994	535,535	74.9	1,364,461
South Dakota	2,414,306	1,367,392	242.3	1,638,946	906,260	173.5	360,716	199,460	48.1	3,074,150
Tennessee	1,039,736	518,939	47.7	1,648,644	824,322	45.0	1,508,780	754,390	47.0	4,445,866
Texas	8,827,775	4,409,623	639.9	10,160,260	5,062,609	458.6	3,325,551	1,571,242	198.8	5,301,352
Utah	686,534	493,723	55.5	1,332,090	949,451	121.9	97,770	69,230	7.9	1,077,280
Vermont	799,081	390,183	23.3	1,747,230	772,795	43.7	92,330	45,969	3.1	45,277
Virginia	2,054,457	1,027,228	107.3	2,824,737	1,356,968	69.9	1,186,836	584,968	36.4	1,960,360
Washington	1,660,432	855,474	72.3	2,718,731	1,424,512	39.3	1,135,160	597,200	5.5	612,992
West Virginia	804,265	406,733	23.1	1,673,601	835,894	43.9	400,792	369,031	14.3	2,081,497
Wisconsin	7,443,878	3,523,739	233.8	4,081,871	2,001,677	118.3	1,344,401	525,500	20.2	1,097,198
Wyoming	2,329,852	1,424,305	235.9	1,356,222	819,929	154.6	167,241	103,140	18.5	179,548
District of Columbia	459,087	226,420	8.0	504,649	248,452	9.9	443,946	202,633	6.9	1,150,702
Hawaii				534,707	265,374	10.7	451,130	225,565	9.4	134,061
Puerto Rico										
TOTALS	134,017,654	69,944,139	5,889.7	172,532,992	86,794,797	5,537.0	71,335,298	33,046,581	2,163.5	90,013,952

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF NOVEMBER 30, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$4,151,115	$3,944,757	$3,903,775	136.9	$ 250,100	$ 243,478	7.8				$3,862
Arizona	2,569,841	3,156,647	2,507,267	193.7	38,548	38,548					24,026
Arkansas	3,352,061	3,150,524	3,123,198	350.5	181,216	180,523	9.7				48,339
California	7,747,928	7,527,537	7,171,405	257.5	572,929	515,656	6.4				60,868
Colorado	3,395,263	2,385,907	2,294,163	99.4	89,597	89,596	6.0	$ 8,200	$ 8,200		1,003,304
Connecticut	1,418,709	1,180,464	1,102,094	17.9	232,031	206,330	4.5	124,130	64,435	0.2	45,850
Delaware	900,310	665,348	639,615	56.6	219,462	219,462	10.3	26,712	26,712	4.4	14,521
Florida	2,597,144	2,598,621	2,529,126	99.1	38,957	38,957					29,061
Georgia	4,988,967	1,819,038	1,777,667	98.4	2,450,184	2,181,804	116.5	442,460	442,460	29.6	587,036
Idaho	2,222,747	2,267,396	2,161,480	185.9	39,559	39,559					21,707
Illinois	8,694,009	8,028,820	7,786,908	456.2	781,010	781,010	32.0				126,091
Indiana	4,941,255	4,872,511	4,578,655	219.7	331,419	331,419	18.7				31,181
Iowa	4,991,664	5,223,196	4,879,904	528.3	110,198	107,765	.3	3,474	3,470		525
Kansas	4,994,975	4,552,153	4,482,802	356.3	419,276	417,103	34.2	15,620	15,620	1.6	79,449
Kentucky	3,726,271	3,465,086	3,307,265	349.8	394,471	394,471	8.7				24,535
Louisiana	2,890,429	2,666,658	2,373,902	165.1	428,080	367,449	2.6	97,087	97,070	10.4	52,008
Maine	1,676,799	1,579,111	1,566,938	72.6	112,460	109,861	3.5				
Maryland	1,750,738	637,876	631,089	23.4	581,095	581,095	11.6	193,134	159,302	6.4	379,253
Massachusetts	3,262,885	2,183,676	2,183,676	18.0	829,930	439,160	.4	1,136,726	568,363	.8	71,686
Michigan	6,301,414	6,520,746	5,948,097	288.6	290,686	289,571	3.3	44,500	38,000	.3	25,746
Minnesota	5,277,145	6,403,838	5,170,514	896.0	96,799	88,962	5.9				17,669
Mississippi	3,457,552	2,984,094	2,979,471	209.8	395,446	394,406	25.9	10,800	10,800	.6	72,874
Missouri	6,012,652	5,294,154	5,144,127	776.9	790,784	736,082	.7	34,390	32,294		100,149
Montana	3,676,416	3,597,216	3,536,508	200.7	95,385	95,385	.1	8,462	8,462		36,062
Nebraska	3,870,739	3,311,029	3,203,989	346.4	587,690	587,686	24.3	70,270	70,270	1.8	8,795
Nevada	2,243,074	2,291,627	2,196,238	110.1	84,970	38,146	1.7				8,690
New Hampshire	945,225	876,181	843,257	37.7	121,092	101,301	5.7				667
New Jersey	3,129,805	1,240,345	1,220,240	23.7	1,874,079	1,874,079	11.7	34,468	34,468		1,018
New Mexico	2,871,397	2,809,901	2,804,735	213.7	43,071	43,071		14,681	12,196		11,395
New York	11,046,377	10,762,211	10,305,153	170.0	349,700	309,700	2.0	69,000	69,000	.2	362,524
North Carolina	4,720,173	4,451,805	4,379,902	267.8	285,491	285,491	22.9	37,900	37,900	.5	16,881
North Dakota	2,867,245	2,493,139	2,452,479	378.9	107,799	107,799	1.2	282,834	282,834	36.0	24,134
Ohio	7,670,815	6,281,178	6,195,424	280.2	1,431,922	1,396,497	16.4	30,200	30,200	1.9	48,694
Oklahoma	4,580,670	4,477,024	4,268,653	396.4	293,970	293,970	12.0	8,800	5,000	.1	13,047
Oregon	3,038,642	3,216,548	2,950,598	164.6	45,580	45,580		11,846	11,846	1.0	30,618
Pennsylvania	9,347,797	5,969,960	5,727,602	235.1	3,521,971	3,075,081	40.2	497,593	388,504	10.6	156,609
Rhode Island	989,208	1,111,600	989,136	18.8							72
South Carolina	2,702,012	2,261,423	2,111,864	224.6	617,029	545,752	24.9	9,664	9,664	1.5	34,731
South Dakota	2,976,454	2,596,836	2,593,052	476.1	369,237	369,237	28.6				14,165
Tennessee	4,192,460	3,190,715	3,139,537	126.1	1,020,913	1,020,913	24.6	35,710	22,060	2.6	9,951
Texas	11,989,350	12,592,804	11,526,736	1,105.8	458,571	346,058	11.9	83,337	83,100	7.2	33,455
Utah	2,067,154	2,119,876	1,904,505	206.7	131,655	131,655	1.6				30,994
Vermont	924,306	1,061,113	915,284	23.2	8,000	8,000					1,022
Virginia	3,652,667	3,377,912	3,260,504	990.8	178,541	173,644	14.3				218,519
Washington	3,026,161	3,313,249	2,936,854	164.3	45,392	45,392		25,465	20,845	.7	23,070
West Virginia	2,231,412	1,970,965	1,876,155	81.0	380,572	322,457	14.5	32,800	32,800	5.0	
Wisconsin	4,823,884	5,240,385	4,727,518	343.4	94,268	93,900	.3				2,466
Wyoming	2,219,155	2,172,672	2,167,016	152.4	33,287	33,287					18,852
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	678,560	655,700	10.4	277,293	215,689	7.0	10,000	10,000		44,644
TOTALS	195,000,000	177,524,432	168,081,273	12,614.3	22,131,715	20,352,037	574.9	3,400,263	2,595,875	123.4	3,970,815

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

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Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

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DEPARTMENT BULLETINS

No. 1279D. .Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D. .Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T. . .Highway Bridge Surveys. 20 cents.

No. 265T. . .Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP. .The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. Roadside Improvement. 10 cents.

No. 272MP. Construction of Private Driveways. 10 cents.

No. 279MP. Bibliography on Highway Lighting. 5 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. .Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF NOVEMBER 30, 1937

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NLW PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,557,534	$ 3,539,035	47	1	12	$ 378,519	$ 378,519	2			$ 175,300	$ 109,000	4			$ 8,063
Arizona	1,256,099	1,211,282	1,162,221	14		1	91,216	68,807	1		3	2,280	2,280			1	22,791
Arkansas	3,574,060	2,823,031	2,815,830	51	6	4	673,549	672,844	5		13	46,324	46,324			16	39,062
California	7,486,362	7,154,760	6,966,713	45	8		470,205	470,204	2			10,000	10,000	5			39,445
Colorado	2,631,567	1,907,899	1,828,075	23	1		821,664	791,096	7	2	21						12,397
Connecticut	1,712,684	297,379	297,379	2	1	1	1,341,801	1,320,440	8								94,865
Delaware	418,239	130,000	130,000	1			277,993	277,993	2								10,246
Florida	2,827,883	2,411,470	2,381,150	30	5	2	119,018	119,018	1			133,770	133,770			73	193,945
Georgia	4,895,949	599,219	597,715	14	5	5	1,386,449	1,386,449	24	7	6	776,040	776,040	11	2	41	2,135,745
Idaho	1,674,479	1,396,104	1,385,766	21	2	9	256,458	256,458	2		9	4,261	4,261			3	27,994
Illinois	10,307,184	8,808,994	8,687,750	65	8		1,516,234	1,509,830	8	2		127,000	78,000	2			31,604
Indiana	5,111,096	4,112,984	3,998,799	37	12		1,108,824	1,108,824	5		162						3,474
Iowa	5,600,679	4,379,206	4,285,042	95	9	8	1,300,569	1,294,781	11								20,856
Kansas	5,246,258	3,955,532	3,932,379	54		5	1,273,119	1,253,704	4	1		115,024	22,020	2		3	38,155
Kentucky	3,672,387	1,358,648	1,350,795	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		14,526
Louisiana	3,213,467	1,328,350	1,328,343	14	1		1,120,878	1,120,859	11	1		649,599	600,110	4	1		164,155
Maine	1,426,861	1,265,372	1,263,094	19	1	4	160,788	142,340	1	1							21,227
Maryland	2,061,751	501,739	501,739	3	3	16	850,670	850,670	6	2	4	476,862	404,393	2	1	10	304,949
Massachusetts	4,210,833	2,488,969	2,486,539	20	4		1,305,790	1,305,790	6			249,991	249,991	1			168,513
Michigan	6,765,197	6,877,565	6,579,490	44	8		101,477	101,477				43,500	43,500		1		40,730
Minnesota	5,395,441	4,776,166	4,677,484	81	14	50	709,458	703,408	5								14,550
Mississippi	3,241,475	2,040,404	2,040,235	45	4	6	690,038	690,038	10	3	3	78,300	78,300	2		14	432,902
Missouri	6,142,153	3,012,558	2,944,928	31	1		3,293,571	3,190,761	18		1	1,650	1,650			1	4,814
Montana	2,722,327	2,588,865	2,532,327	37	7		245,576	185,931	1	1							4,069
Nebraska	3,556,441	2,810,180	2,774,426	74	3	19	554,167	554,167	7		1	203,027	203,027	3		4	24,821
Nevada	887,260	886,182	857,417	8	3		13,308	13,308				3,630	3,630			5	12,905
New Hampshire	822,484	737,902	737,857	8	4	6	106,670	82,521	2			2,106	2,106			1	
New Jersey	3,983,826	3,058,834	3,046,907	20	5	1	831,570	831,570	4	4		44,730	44,730		1		60,619
New Mexico	1,725,286	1,644,472	1,637,584	18	1		82,384	82,384	1								5,318
New York	13,577,189	10,476,925	10,182,856	33	39		3,107,900	3,107,900	12	10		94,000	94,000				192,433
North Carolina	4,823,958	3,439,944	3,419,818	46	18		1,298,009	1,298,009	16		115						106,130
North Dakota	3,207,473	2,819,708	2,815,995	56	4		389,012	389,012		1							2,466
Ohio	8,439,897	2,110,287	1,990,344	15	5	2	5,875,528	5,407,381	32	3	2	866,660	851,660	12		10	190,512
Oklahoma	5,004,711	3,668,274	3,656,660	57	9	7	1,356,324	1,223,224	8	3	10	75,190	75,190			24	49,637
Oregon	2,334,204	2,329,684	2,242,111	16	6	2	74,215	74,215	1								17,878
Pennsylvania	11,483,613	7,590,688	7,077,760	64	18	9	4,335,166	3,968,178	22	5		150,473	150,473	5			287,202
Rhode Island	699,691	700,500	695,685	4	3												4,006
South Carolina	3,059,956	1,567,463	1,548,229	31	11	9	1,013,267	1,005,837	15	4	23	192,517	192,517	1	1	16	313,373
South Dakota	3,249,086	2,559,360	2,559,162	53	6	53	473,336	473,336	12			269,350	210,393	17			6,195
Tennessee	3,903,979	1,305,258	1,296,577	25	3	26	2,212,100	2,212,100	20	1	2	151,380	151,380	1		11	243,922
Texas	10,855,982	9,803,489	9,792,408	125	14	77	552,216	552,216	3	1	12	182,969	182,969			60	328,389
Utah	1,230,763	1,118,408	1,111,512	15	1	1	103,523	103,523	2								15,728
Vermont	729,857	749,535	705,319	10	7	18	14,100	14,100			2						10,438
Virginia	3,774,287	2,496,532	2,382,064	42	15	15	1,341,858	1,335,233	10	5	3	40,980	38,966		1	6	18,024
Washington	3,095,041	2,719,882	2,696,786	22	11	11	368,494	368,069	1	1							30,186
West Virginia	2,677,937	862,164	860,198	8	1	4	1,735,439	1,734,936	18	2	2	78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,017,481	3,988,449	37	5		923,521	922,766		2	6	123,559	61,711			27	49,756
Wyoming	1,360,841	1,161,714	1,152,337	12			189,365	189,365	2								19,139
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			226,162	158,370	2								11,327
TOTALS	196,000,000	140,322,066	137,636,098	1617	298	383	48,655,717	46,996,648	335	68	400	5,981,906	5,513,825	77	9	327	5,853,429

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 18, NO. 11 ▽ JANUARY 1938

THE ANGEL OF SHAVANO, IN THE COLORADO ROCKIES

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Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Volume 18, No. 11 January 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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PRESENT KNOWLEDGE OF THE DESIGN OF FLEXIBLE PAVEMENTS[a]

A REVIEW OF EXISTING DATA AND DISCUSSION OF NEEDED RESEARCH

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by A. C. BENKELMAN, Associate Research Specialist

DURING recent years highway engineers have become more and more interested in the problem of the design of flexible pavements. The reason for this interest is two-fold: First, because of the large mileages of this type of pavement that are built annually and the desire to build such pavements according to scientific principles; and second, because until accepted principles of design are formulated neither the question of the relative economy of different types of road surface nor the allied question of allocation of vehicle costs can be answered authoritatively.

The purpose of this report is to review the status of our existing knowledge of the problem and to point out what additional information is needed in order that dependable principles of design can be formulated. The report first reviews what is known about the design of the component parts of the road structure, continues with a discussion of the methods of design or formulas that have been suggested for determining the thickness of road surfaces, and concludes with a brief statement of the research aspects of the problem.

The statement has been made that the function of a pavement of the flexible type is to distribute the wheel load to the subgrade in such manner that the intensity of pressure will cause neither permanent nor elastic deformation of the soil of sufficient magnitude to produce failure of the pavement surface.[1] This means that primarily the subgrade and not the surface constitutes the medium of support of vehicular loads. Thus, knowledge of a quantitative character concerning the bearing capacity of soil materials when subjected to forces identical with those created by traffic loads is a first essential for the development of basic principles of flexible pavement design.

Great progress has been made during recent years in the study of soil science as related to the design and construction of highways. Certain basic facts established prior to 1930 led to the development of ways and means for improving the stability of subgrade soils. For example, it was known that (1) the performance of a subgrade is governed by the physical characteristics of the soil, i. e., cohesion, internal friction, compressibility, elasticity, and capillarity, (2) these characteristics depend upon soil constituents which may be easily identified by laboratory tests, and (3) subgrades may be classified into groups on a basis of their physical characteristics.

SEVERAL METHODS CURRENTLY USED TO STABILIZE SOILS

In 1929 Hogentogler and Terzaghi [2] suggested an approximate method of analysis with which it was possible to develop some interesting data regarding the probable effect of the characteristics and condition of the soil on bearing power. They indicate in part that by combining a cohesive clay with a granular soil in the proper proportions, its bearing capacity could be more than tripled and, also, that the supporting capacity of a wet cohesive soil might be as low as one-thirtieth of the bearing capacity of the same soil when dry. The authors were able to demonstrate that the unit bearing capacity of a noncohesive soil increases with increase in the width of loaded area, and that the amount of increase is different in different soils. For soils entirely lacking in cohesion, computed values indicated that a marked increase in unit supporting capacity would accompany an increase in width of loaded area, whereas for cohesive soils no increase would be expected.

Methods employed at the present time for improving the stability of subgrade soils and foundation course soil mixtures are predicatd to a large extent upon the fundamentals enumerated above. They consist primarily of (1) soil stabilization wherein granular materials are added to cohesive soils or cohesive binder is incorporated with granular soils, (2) treatment with different materials such as deliquescent chemicals, bituminous products, or portland cement, and (3) installation of drainage structures or the construction of surface treatments which separately or jointly function to prevent cohesive soils from taking up moisture.

Recently other means have been developed for improving the stability as well as the permanency of support of subgrade soils. The work of R. R. Proctor [3] for example, indicates that for every soil there is an optimum moisture content at which maximum density can be obtained with a given method of compaction; and studies by Hans F. Winterkorn [4] point to the fact that the physical properties of a subgrade may be altered completely by chemical treatment of the soil.

The question of changing the characteristics of soils by artificial means has been touched upon here merely for the purpose of indicating that it is possible, on a basis of existing knowledge, to improve the quality, uniformity, and permanency of support of subgrade soils. The fact that the subgrade or the foundation course, which is generally composed of soil mixtures, plays such an important role in the performance of a flexible pavement emphasizes the need of designing these component parts of the structure in such manner

[a] Paper presented at the Seventeenth Annual Meeting of the Highway Research Board, Washington, D. C., 1937.

[1] A Symposium on Research Features of Flexible-Type Bituminous Roads, by E. F. Kelley. Part II. Proceedings of the Fourteenth Annual Meeting, Highway Research Board, 1934.

[2] Inter-relationsbip of Load, Road and Subgrade, by C. A. Hogentogler and Charles Terzaghi. PUBLIC ROADS, vol. 10, no. 3, May, 1929.

[3] Fundamental Principles of Soil Compaction. Engineering News-Record, vol. 111, no. 9, 1933.
Field and Laboratory Determination of Soil Stability. Engineering News-Record, vol. 111, no. 10, 1933.
Field and Laboratory Verification of Soil Stability. Engineering News-Record, vol. 111, no. 12, 1933.
New Prineiples Applied to Actual Dam Building. Engineering News-Record, vol. 111, no. 13, 1933.

[4] Oiling Earth Roads, Application of Surface Chemistry. Industrial and Engineering Chemistry, vol. 26, 1935.
Adsorption Phenomena in Relation to Soil Stabilization. Proceedings, Fifteenth Annual Meeting, Highway Research Board, 1935.
Surface Chemical Factors Influencing the Engineering Properties of Soils. Proceedings, Sixteentb Annual Meeting, Highway Research Board, 1936.

that they will provide uniform and constant support for the surface to be laid upon them. It is believed that this should constitute the basic procedure in design rather than to attempt to compensate for inequalities in foundation or subgrade support by varying the thickness of the surface crust. What appears to be needed at the present time is to translate the qualitative means available for improving subgrade support into quantitative values suitable for use in a mathematical treatment of the problem.

Rolling a Surface-Treated Road Serves to Compact the Surface Into a Dense Mass, Increasing Its Load-Carrying Capacity.

BEARING CAPACITY OF A SOIL DEPENDS UPON SEVERAL VARIABLES

Most of the existing test data on soil bearing power have been developed from studies of the design of foundation footings. Here, in marked contrast to subgrades for flexible pavements, the pressures imposed upon the soil medium are of the sustained type and are of practically uniform intensity. Whether either the test data or the theoretical conceptions of bearing power that have been developed from these studies are applicable at all to the problem under discussion depends, in part at least, upon whether the static wheel load proves to be the critical load for which the thickness of pavement surface must be designed.

If it is found that the action of the rolling wheel load plus the impact effect of a motor vehicle is, under average conditions, more severe than that of the static wheel load, then it would appear that our present knowledge of soil-bearing power has little application in the design of flexible pavements. If, on the other hand, future investigations should show that the combined effect of the moving wheel and impact is less severe than that of the static load alone, then these data should prove of value and the logical procedure would be to base the design or thickness of flexible surfacings upon the static wheel load.

This question is one of the most controversial issues that has entered into attempts to develop a rational method of design on a basis of our existing knowledge. In some of the suggested methods of design there is an allowance for the dynamic effect of moving vehicles of as much as 50 percent over and above the permissible static wheel load, whereas in others no allowance whatever is included.

Space does not permit, nor do the circumstances appear to warrant, a detailed review in this report of the existing data on soil bearing power. An attempt will be made, however, to discuss the problem in a general way and to cite some of the more significant results of the many investigations and analyses that have been made.

It was indicated previously in connection with the approximate method of analysis by Hogentogler and Terzaghi that the bearing capacities of soil media depend upon the size of the loaded area, and that the degree of this dependence is determined by the characteristics of the material. The question of variation in bearing capacity with size of loaded area is an important one, as far as flexible pavements are concerned, because the areas of subgrade pressure lie within the size range in which, according to existing test data, the most marked variations in bearing capacity are encountered.

A great number of investigations have been made concerning the bearing capacities of soils of a cohesive nature. Most of them involved the application of sustained loads to the soil material through rigid bearing plates which ranged in size generally from a few square inches up to 9 square feet. The results obtained, including those of Stötzner in 1919,[5] A. Bijls in 1923,[6] the Bureau of Public Roads in 1925 [7] and Fritz Emperger in 1926,[8] indicate that the unit bearing capacity of such soils for a given settlement decreases as the size of the loaded area increases and that settlement under the same unit load varies almost directly as the square root of the area. W. S. Housel in 1929 [9] in a series of tests that involved several types of cohesive soil found also that for a given settlement the larger the area the smaller the unit bearing capacity. In general, as far as the effect of size of loaded area is concerned, the data he obtained are in agreement with those of the other investigators mentioned above.

In Germany the bearing capacity of soils has been studied quite extensively in recent years. While the majority of tests made have involved soils which, as far as it has been possible to ascertain, might be said to fall on the border line between those of a cohesive and noncohesive nature, the results are of considerable interest. The conclusions reached in these researches indicate, in general, that (1) the unit bearing capacity varies with the size of loaded area, (2) settlement consists of both elastic and permanent compression of the material, and (3) lateral displacement of the soil from beneath the loaded area, particularly in small-sized areas in conjunction with excessive unit pressures, is an important contributory factor to settlement.

EFFECT OF SIZE OF LOADED AREA ON BEARING CAPACITY STUDIED

E. W. Goerner,[10] working in the soils laboratory of the Freiberg Institute under the direction of Dr. Kögler, studied in great detail the load reaction char-

[5] Erzielung gleicher Fundamentsenkung durch wahl des kleineren Bodencinheitdruckes bei der grössern Fundamentfläche. Dissertation, Braunschweig, 1919.
[6] Le Genie Civil, May 1923, pp. 490-492.
[7] The Supporting Value of Soil as Influenced by the Bearing Area, by A. T. Goldbeck and M. J. Bussard. Public Roads, vol. 5, no. 11, January 1925.
[8] Die zulassige Belastung des Baugrundes. Die Bautechnik, vol. 4, 1926.
[9] A Practical Method for the Selection of Foundations Based on Fundamental Research in Soil Mechanics. Department of Engineering Research, University of Michigan. Bulletin No. 13, 1929.
[10] Über den Einfluss der Flächengrösse auf die Einsenkung von Gründungskörpern. Geologie und Bauwesen, no. 3, 1932.

acteristics of a natural sand soil. Static loads were applied to material held in box containers, through circular, rigid bearing plates which ranged in area from 5 square centimeters to 1 square meter (0.775 sq. in. to 10.8 sq. ft.). The containers were of sufficient size and depth to eliminate confining effects. The material in an artificially dried state was placed in the containers with a given degree of compaction for each individual test.

A great number of tests were made, the results indicating rather definitely that the effect of size of loaded area depends, among other things, upon the unit pressures imposed on the material. For very low unit pressures settlement was practically independent of size of area, irrespective of the density of the material. For larger unit pressures, beginning with an area of approximately 75 square inches, the settlement increased as the bearing area was either decreased or increased. According to Goerner, the behavior in the first instance was caused by lateral displacement of the material from beneath the loaded area and, in the second, chiefly by vertical compression which increases as the size of the area increases. The rate of increase of settlement with decreasing area is rapid for, as the area approaches a minimum size, "pile action" apparently develops. The rate of increase of settlement with increasing size of area is more gradual, the data in general indicating that the increase varies almost in direct proportion with the square root of the loaded area.

Goerner's tests were made in the laboratory under carefully controlled conditions. As far as the effect of size of bearing area is concerned, the results as described were definite and conclusive. Heinrich Press [11] in studying the bearing capacity of soils in the field obtained results that are in close agreement with those of Goerner. Most of his work was done in connection with the design of foundations, the tests of bearing capacity of a number of types of soil being followed with observations of settlement of the structures erected. In general, the results of these studies indicate that the settlement of structures cannot be predicted with any great degree of certainty from the behavior of small-sized test areas, particularly for structures resting on soils of a sandy nature.

Recently the Bureau of Public Roads has investigated the bearing capacity of a silt loam soil beneath rigid bearing plates ranging in size from about 3 square inches to 38 square feet, the latter area being nearly four times larger than that used in any of the tests mentioned above. The tests indicate clearly that for areas less than about 5 square feet, the settlement under the same unit load was proportional to the diameter or the square root of the area. For areas exceeding this size, however, the results of the tests showed the settlement to be independent of the size of the bearing area.

The tests were made in connection with an investigation of concrete pavement design, their purpose being to determine the modulus of subgrade reaction in order that measured slab stresses might be compared with those computed by means of Westergaard's theory of slab stress. Because rigid pavement slabs are subject to but very small deflections, the tests were limited, in general, to settlements of a magnitude considerably less than those of the other studies enumerated.

[11] Baugrundprobebelastungen, ihre Auswertung und die au den Bauwerken gemessenen Setzungen. Die Bautechnik, vol. 10, no. 30, 1932.

RELATIONS BETWEEN SETTLEMENT AND SIZE OF LOADED AREA GIVEN FOR COHESIVE AND NONCOHESIVE SOILS

From what has been presented in the preceding paragraphs, it is apparent that our knowledge of soils has advanced to the point where the bearing power of subgrades can be improved in a scientific manner. By the incorporation of granular materials in clay soils or by furnishing granular soils with cohesive binder, they can be improved with respect to both the quality and uniformity of support. By treatment with deliquescent chemicals, the moisture content of graded soil mixtures can be controlled and their supporting power maintained within fairly narrow limits. Soils of defective grading may also be improved by admixtures of bituminous materials or portland cement. In addition to these methods, which are in general use at the present time, the more recent discoveries involving compaction of soils at their optimum moisture content and base exchange by chemical treatment offer further possibilities for improving the support of subgrade soils.

The preponderance of existing test data on soil bearing power indicate that (1) the settlement of cohesive soils under the same unit load varies directly as the square root of the area, and (2) the settlement of noncohesive soils under the same unit load for very low pressures is independent of size of area. As the unit pressure on noncohesive soils is increased, beginning with an area of approximately 75 square inches, settlement increases as the size of area either decreases or increases from this value; as the area increases, settlement increases almost in direct proportion to the square root of the area; as the area decreases settlement increases, at first gradually, then rapidly.

The above generalization relating to cohesive soils is applicable only for pressures which lie well within the safe bearing capacity limits of the material and for areas up to approximately 9 square feet, the maximum size of area used in the majority of tests made. The generalizations for noncohesive soils apply for areas up to about 1 square meter (10.8 square feet).

The test data developed recently by the Bureau of Public Roads in connection with the concrete pavement studies are in disagreement with the above-stated relationship between settlement and loaded area for cohesive soils. In these tests, settlement under low unit pressures, for areas up to about 5 square feet, varied directly as the square root of the area; and for areas above this size up to a maximum of 38 square feet, resistance was indicated to be independent of size of area. Inasmuch as these tests were made with care and precision, there is no reason to doubt that the results reflect the true area-settlement reactions of this particular soil medium. Why the results fail to agree with the other test data cited to the extent above indicated is not known.

Obviously the true influence of the size of bearing area for a particular grade of soil cannot be determined by test unless the material in question is perfectly homogeneous to a depth at least equal to that to which the effect of the load extends. It is the exception rather than the rule to find such homogeneity in soils as they exist in nature, and it is largely for this reason that the results of bearing-capacity experiments on areas of test size cannot always safely be extended to pressure areas of a size such as are found beneath foundation footings or even rigid pavements.

Beneath flexible pavements, pressure areas on the subgrade resulting from loads on the surface will tend

to be small and, in general, may not exceed 10 square feet. This does not mean, however, that the penetration-area relationship suggested by the preponderance of existing test data is applicable to the problem of design of such pavements. In the first place, the test data cited were developed using rigid bearing plates; and in the second place, with possibly one exception, the material surrounding the loaded area was not restrained from movement in any way. Conditions are quite different in flexible pavement subgrades because the wheel load is transmitted to the soil through a flexible medium whose weight serves to increase the resistance of the soil to lateral as well as upward movement.

FEW DATA AVAILABLE ON PRESSURE DISTRIBUTION UNDER FLEXIBLE PAVEMENTS

Few attempts have been made to develop comprehensive data on either the load reaction characteristics of flexible pavements or the manner in which they distribute loads to the subgrade. The reason for this probably lies in the difficulty of developing satisfactory apparatus and testing technique rather than any lack of interest in the subject. However, even with the somewhat crude methods employed at the time of the Bates Road tests [12] and by the Bureau of Public Roads in 1923,[13] information of interest and of value was produced.

When the Bates road was built, a total of 26 pressure cells were installed on the subgrade of certain of the sections. Tests were made subsequently, involving study of the deflection of the surface and pressure distribution on the subgrade as a loaded truck was moved slowly toward and away from the points where the cells were located. While only a single cell was installed on the subgrade beneath a flexible pavement section (sec. 5—3-inch brick, 2-inch mastic cushion, 8-inch rolled stone base) data obtained during the limited tests made are of interest.

Before subjecting the test road to truck traffic, a 4-ton wheel load produced a maximum pressure of 12 pounds per square inch on the subgrade. After 1,000 trips of the 2,500 pound wheel-load increment of loading the pressures produced by the 4-ton static wheel load amounted to 30 pounds per square inch. Apparently the mechanical bond existing in the rolled stone base course had been at least partially destroyed by the disruptive action of the moving trucks. This would decrease the area over which the static wheel load was distributed to the subgrade and thus increase the unit pressure.

The tests by the Bureau of Public Roads in 1923 involved the application of sustained loads, through a bearing block shaped to simulate a truck wheel, to layers of crushed stone of several thicknesses laid both upon a dry sand and a wet clay supporting medium. The distribution of the loads through the materials was measured with pressure cells placed both on and beneath the soil layers. These tests indicated that (1) under a given load the maximum intensity of pressure decreases as the thickness of surface increases, the decrease being, roughly, directly proportional to the increase in thickness; (2) in general, the thicker the surface the higher the load-carrying capacity; (3) the thicker the surface, the higher the intensity of pressure on the subgrade can be, at least until failure begins to take place; and (4) the more resistant the supporting medium, the higher the intensity of pressure developed and the greater the ratio between the maximum and average pressures.

In 1924 the Bureau of Public Roads [14] studied the bearing capacity of a number of gravel roads in Minnesota. The surfaces ranged in thickness from 6 to 12 inches and all were on the same general type of subgrade. Static loads were applied to the surface crust through a rectangular-shaped bearing block, 52 square inches in area. The tests indicated that:

1. Settlement was almost directly proportional to load.

2. Resistance of the surfaces was high; four out of six locations supported a load of 100 pounds per square inch at a settlement of 0.10 inch.

3. The effect of the loads was confined practically to the area beneath the bearing block, deflection gages placed 6 inches away registering no movement.

4. At one location after loading the surface the gravel course was removed and the subgrade soil loaded directly. For equal settlements up to 0.10 inch the subgrade supported only about one-half the unit load carried by both the surface (6-inch thickness) and the subgrade.

Tests made in 1928 [15] by the Bureau of Public Roads throw some light on the possible difference in impact reaction of flexible as compared to rigid pavements. It was observed that a flexible pavement consisting of a 1-inch sheet asphalt wearing course on a waterbound macadam base deflected over an appreciable area under an impact load and then sprang back without apparent injury. The deflection at the point of application of the load amounted to as much as ½ inch, some deflection being noticeable at a point 3 feet away. In several rigid pavements, which were subjected to the same test, the deflection was much smaller and the impact reaction higher.

VARIATION IN PRESSURE UNDER BEARING PLATES OF DIFFERENT RIGIDITY DISCUSSED

A great amount of investigation and study has been given to the problem of pressure distribution in soils in connection with the design of foundation structures. Here pressures are transmitted to the soil through bodies which, in contrast to flexible pavement surfaces, are essentially rigid in character. Some work carried on in Germany indicates the difference in the pressure distribution as well as the supporting capacity of soils when loaded through bearing plates whose degree of rigidity was varied between rather wide limits.

Kögler and Scheidig [16] in a series of tests studied the pressure distribution in sand when sustained loads were applied to the material through three bearing plates, centrally loaded. All were 99 centimeters (39 in.) in diameter, but varied greatly in rigidity. At a depth of 40 centimeters (15.75 inches) the maximum pressure amounted to 103, 153, and 181 percent of the applied unit pressure for rigid, semirigid, and elastic plates, respectively. At a depth of 10 centimeters (3.94 inches) the maximum pressure amounted to 160 and 300 percent of the applied unit pressure for the rigid and

[12] Bates Experimental Road, by C. Older. Bulletin No. 18. State of Illinois, Division of Highways, 1924.

[13] Highway Engineering Investigations at the Arlington Experiment Farm, by A. T. Goldbeck. Proceedings Ninth Annual Conference on Highway Engineering, University of Michigan, 1923.

[14] Unpublished data of the Bureau of Public Roads.

[15] Effect of Pavement Type on Impact Reaction, by J. T. Thompson, PUBLIC ROADS, vol. 9, no. 6, August 1928.

[16] Druckverteilung im Baugrunde, F. Kögler and A. Scheidig. Die Bautechnik. No. 31, July 15, 1927.

flexible plates, respectively, no value being given for the semirigid plate. As was to be expected, the greater the rigidity of the bearing plates the less the variation in unit pressure under the plates.

Additional information on the same subject was developed by Heinrich Press in 1934.[17] Sustained loads were applied to two types of soil through square bearing plates of different rigidity, the pressure being recorded in the material at depths of 10, 32, 54, and 76 centimeters (3.94, 12.6, 21.3, and 29.9 in.). The vertical displacement of the plates was recorded also, both at the edge and center. One of the soils was the same as that used in the tests by Kögler and Scheidig, the other being described as a loam containing 21 percent sand. Tests were made on the latter material both in the dry state and when containing 32 percent moisture.

The results of the tests are summarized in table 1. These data indicate clearly that the maximum pressure that may develop at some depth in a soil from the application of an external load depends to a marked degree upon the flexibility of the body through which the load is transmitted.

TABLE 1.—*Results of sustained load tests on different soils using bearing plates of different rigidity*

Soil	Maximum soil pressure 10 cm (3.94 in.) beneath surface expressed in terms of applied unit load					
	60 by 60 cm (23.6 by 23.6 in.) plates			50 by 50 cm (19.7 by 19.7 in.) plates		
	Rigid	Flexible—1.5 mm (0.059 in.) steel		Rigid	Loaded at center	
		Loaded at center	Loaded uniformly		Semirigid (0.0787 in.) 2.0 mm steel	Flexible (0.059 in.) 1.5 mm steel
Sand	1.36	3.05	1.50	1.50	2.95	3.30
Loam, dry	1.35	2.90	1.09	1.49	2.80	3.14
Loam, moist	1.26	3.00	1.50	1.50	2.98	3.20

USE OF OSCILLATING MACHINE IN STUDY OF PAVEMENTS DESCRIBED

With regard to the displacement or settlement of the soils, only limited data in graphical form are given in the report. In case of the rigid 60- by 60-centimeter bearing plate on the sand, the settlement amounted to about 0.2 centimeter, and for the flexible plate loaded at the center with the same apparent unit load it amounted to approximately 2 and 1 centimeters at the center and at the edge, respectively. For the rigid plate on the dry loam, the settlement was about 0.3 centimeter and for the centrally loaded flexible plate the settlement at the center was about 1.5 centimeters. For the flexible plate, uniformly loaded on the sand, the average settlement was about 0.7 centimeter and on the moist loam about 0.5 centimeter. It is of interest to note from the graphical data given in the report that the sand beneath the rigid plate supported with considerably less settlement a unit load 2.5 times greater than that which it supported when beneath the flexible plate. Comparable data for the loam soil are not given.

Recently a comparatively new method of test has been used in Germany to study the load reaction characteristics of soils and road surfaces. The method, briefly described, involves the application of periodic forces, the intensities of which vary according to a sine law, to a material and the study of its physical reaction by measurements of displacement and speed of propagation. An oscillating machine is used to apply the forces. It consists essentially of two parallel shafts that rotate in opposite directions and to each of which is attached an eccentric weight. The weights are so synchronized that, as they revolve, only vertical reactions are developed.

The displacement of the soil medium or road surface under the action of this machine is measured at successive distances from the center of force application by means of precise, electrically operated seismographs. The same instruments are used also to record the rate at which a given force impulse is propagated in the medium.

A great amount of work has been done in the development and standardization of this method of test. First, it was used for studying the effect of vibrations in buildings and other structures and later for investigating the physical properties of soils. During the past few years, it has been used to study the properties of both flexible and rigid pavements when acted upon by dynamic forces.

One investigation of unusual interest was carried on at Stuttgart, Germany, under the direction of F. J. Meister.[18] The purpose of the investigation was primarily to evaluate the intensity of the forces imposed upon flexible pavements by moving vehicles. To do this it was first necessary to develop quantitative values that would indicate the ability of such pavements to support loads of a dynamic nature. This information was developed with the oscillating machine and displacement recording instrument. The machine was operated at different frequencies and at different load intensities on the surface of the road and the displacement of the pavement in a vertical and in two horizontal directions was measured.

Tests of this general nature were made on four flexible pavements and on one rigid pavement. Subsequently trucks, both empty and loaded, equipped with different types of tires were operated at various speeds over the surface of one of the flexible pavements and the displacement of the surface again recorded. The displacement values were then converted to kilograms of dynamic force.

EFFECT OF DYNAMIC LOAD MEASURED BY OBSERVING PAVEMENT DEFLECTIONS

Inasmuch as the size and contact area of the tires used in the moving load tests varied over rather wide limits, it was necessary in the preliminary tests with the oscillating machine to investigate the effect of size of the contact area on which the dynamic load was applied. The results obtained from a limited series of tests involving the application of the same force intensity at several frequencies through the normal machine contact area (1,220 sq. cm, 78.7 sq. in.), and through an area about one-tenth of this size indicated no consistent or appreciable difference in effect.

It is true that one might well question whether the effect of loads of either a static or dynamic nature, on flexible pavements, can be measured accurately by displacement observations made either at the point where

[17] Druckverteilung unter starren und elastischen Lastflächen verschiedener Grosse bei verschiedenartiger Belastung im Sand und Lehn. Die Bautechnik, No. 43, October 5, 1934.

[18] Die dynamischen Eigenschaften von Strassen. Dissertation. Martin Boerner, publisher, Halle (Saale), Germany. 1936

the load is applied or at some distance from this point. In the above-mentioned tests the conclusions as to relative effect were based on the vertical displacements measured at a distance of 0.4 meter from the point at which the load was applied.

Two separate and distinct series of tests using moving loads were made, one with the trucks traveling over the natural surface and the other with the wheels on one side of the vehicle passing over an obstruction 1 inch in height at the point where the displacement of the surface was recorded. The results of the tests, which are shown both in graphical and tabular form in the report, indicate in general that (1) on a flexible pavement of normal and uniform smoothness, the intensities of the dynamic loads vary in almost a direct proportion with vehicle speed, (2) the magnitude of the sprung load has very little effect upon the intensity of the resulting force.

When the artificial obstruction was employed the data indicate that for a given tire type the same dynamic effect was developed with a wheel load of 2,100 kilograms (4,620 pounds) as with one of 1,350 kilograms (2,970 pounds). The indicated dynamic force was 2,500 kilograms (5,500 pounds) for both wheel loads when solid-rubber-tire equipment was used and 1,000 kilograms (2,200 pounds) with pneumatic tire equipment.

With the truck traveling over the normal road surface, the same wheel loads equipped with solid tires produced dynamic forces of 575 kilograms (1,265 pounds) and 500 kilograms (1,100 pounds), respectively, while with pneumatic tires the indicated forces were 250 kilograms (550 pounds) and 175 kilograms (385 pounds), respectively.

It is difficult to reconcile the force values obtained by this method of test with the results of the motor-vehicle impact studies made in this country. In this connection Meister is not specific in stating whether his dynamic force values are to be interpreted as representing the dynamic increments alone or the total forces developed by the movement of the vehicles over the road surface.

Earlier in this report attention was called to the fact that as yet we do not know whether the thickness of flexible pavements should be based on the static or dynamic wheel load. Not only is adequate information lacking concerning the intensity of the dynamic wheel load on the average flexible pavement but we know very little of the effect such a load has upon the structure as compared to the static wheel load. According to Meister, it would be a simple matter to develop quantitative data regarding the ability of flexible pavements to support moving vehicular loads by means of the oscillator method of test. He states that, "* * * for low frequencies of vibration or low rates of load application, the load-displacement values constitute an index of the dynamic modulus of pavement reaction * * *;" i. e. the force necessary to displace 1 square centimeter of the surface to a depth of 1 centimeter.

Certainly the development of such data by any method of test for pavements of various thicknesses laid upon the same and different subgrades would prove of great value in establishing basic principles of flexible pavement design.

STRESSES INDUCED IN PAVEMENT BY OSCILLATING MACHINE COMPUTED

Dr. A. Ramspeck, at present a member of the staff of the German Research Society of Soil Mechanics, has carried on some work with the oscillator method of test which suggests the possibility of developing information of interest and value concerning the dynamic properties of concrete pavements.

One of the first facts established in studies of the dynamic properties of soils with an oscillating machine was that the rate at which a quickly applied force is propagated in a soil medium depends upon the elastic properties and possibly upon other physical characteristics of the soil. Such a force sets up vibrations in the medium, the propagation of which takes place in the form of waves whose length is a function of the speed of movement, low speeds being characterized by short wave lengths, high speeds by long wave lengths. From this it would appear that the higher the propagation speed of dynamic waves in road structures, the less severe the vibratory disturbances would be.

The tests by Ramspeck[19] were predicated in part upon these considerations. Besides investigating the rate at which oscillator force impulses were propagated in concrete pavements of various thicknesses laid upon different types of subgrade, he measured the amplitudes of slab movement at different points. From the data obtained he developed the deflection profiles of the slab existing at any given instant. Since an oscillating machine was used to apply the forces, he assumed the vibrations were sine-shaped and on this assumption developed mathematical relations by means of which the bending stresses in the concrete were computed.

In one set of tests, Ramspeck, in the manner described above, computed the vibratory bending stress in slabs of the same thickness (25 cm.) laid upon three types of subgrade: (1) Poor-quality sand; (2) loam soil on which a sub-base of crushed stone 7 centimeters thick was placed, and (3) high-quality rolled gravel. The same force intensity was applied to all three slabs at a frequency of 25 cycles per second. The computed values of slab stress as given in the report amounted to 0.074, 0.014 and 0.007 kilogram per square centimeter (1.0, 0.20, and 0.10 lb. per sq. in.), for bases (1), (2) and (3), respectively.

In connection with these tests, sine wave profiles of the slabs, reproduced from the seismograph records, are shown in the report. These are interesting inasmuch as they show the differences in wave length and amplitude of movement that were observed with the three test slabs. The wave length for the test slabs on subgrades (1) and (2) was about 10 meters, while that of the slab on subgrade (3) was in excess of 20 meters. The amplitude of vibratory movement, in contrast to the wave lengths, was nearly the same for slabs on subgrades (2) and (3) (0.0010 to 0.0012 mm), while for the slab on the poor sand subgrades the amplitude was 0.006 millimeter.

OSCILLATING MACHINE USEFUL IN PRODUCING REPEATED DYNAMIC FORCES OF KNOWN MAGNITUDE

Ramspeck also investigated the action of joints in concrete pavements when the structure was set in vibration with an oscillating machine. In one series of tests he measured the amplitude of movement at six points, three on one side of a longitudinal joint and three on the other, the slab in question being laid upon a compacted gravel subgrade. The joint was of the dummy type, a groove 6 centimeters deep being cut into the surface with a special joint-cutting machine.

[19] Dynamische Untersuchungen auf Strassendecken. Die Beton-Strasse, No. 2. 1936.
Praktische Anwendungen dynamischen Untersuchungen Veröffenentliehungen des Instituts der Deutschen Forschungsgesellschaft fur Bodenmechanik (Degbo) an der Technischen Hochschule, Berlin. Heft. 4. Julius Springer, publisher. 1936.

The presence of this groove in the surface did not in any way serve to disturb the continuity in vertical movement of the slab. In another series of tests the vertical displacement of a pavement laid on a yielding subgrade and containing transverse joints was measured out to a distance of 60 meters from the point where the force impulses were applied. In these tests the continuity of the displacement or deflection curve was broken at each of the joints.

Ramspeck explains these phenomena as follows: A subgrade whose elastic properties or physical characteristics tend to approach those of the surfacing material will vibrate in harmony with the surface and the two media will tend to respond as a unit to vibratory motions. On the other hand, a subgrade whose physical characteristics differ to a marked degree from those of the surfacing material is apt to vibrate out of harmony with the surface. In other words, a sharp plane of discontinuity, at least as far as vibratory movements are concerned, may or may not exist between the road surface and supporting medium.

By studying the rate at which oscillator force impulses are propagated in road bodies before and after laying concrete pavements, information was developed which, according to Ramspeck, may be used to indicate the thickness of slab necessary on new construction work. For example, according to graphical data shown in his report, a slab 25 centimeters in thickness would be necessary on a subgrade having a propagation speed of 125 meters per second (410 ft. per sec.), one 20 centimeters in thickness for a speed of 210 meters per second (690 ft. per sec.), and one 15 centimeters in thickness for a subgrade whose propagation speed is 250 meters per second (820 ft. per sec.).

In connection with these particular data, Ramspeck infers that it should be possible to develop information concerning the proper thickness of pavements of the flexible types for different subgrades from measured values of propagation speed.

The above constitutes only a very brief résumé of the dynamic method of test. The results of certain investigations have been cited not because they are considered highly significant or conclusive, but primarily for the purpose of illustrating the type of information concerning the dynamic and vibration properties of road structures that may be developed with this method of test.

The fact that repeated dynamic forces of known magnitude and characteristics can be applied to a road structure with an oscillating machine is one of the strongest arguments in favor of this method of testing. It should be possible with such a machine to apply forces that are similar in character to those which are imposed upon pavements by moving vehicles inasmuch as the revolving eccentric weights are analogous to the unsprung mass of a vehicle and the weight of the machine is analogous to that of the sprung load. The obvious advantages gained from the use of such a machine in pavement design studies instead of moving vehicles are: (1) The magnitude of the forces can be definitely controlled and determined, and (2) the same intensity of force can be applied and repeated at any desired point and at a range of rates of application.

FORMULAS FOR COMPUTING THE THICKNESS OF FLEXIBLE PAVEMENTS DISCUSSED

As early as 1901, consideration was given to the possibility of designing flexible pavements according to scientific principles. At that time what is now known as the Massachusetts' rule was evolved. This rule states that "the pressure is distributed through macadam and any ordinary gravel or boulder base at an angle of 45° with the horizontal, and the resulting maximum pressure (P) on the subgrade amounts to the wheel load (W) divided by the square of twice the thickness (T) of pavement." This rule may be expressed algebraically as follows:

$$P=\frac{W}{(2T)^2} \text{ or } T=0.5\sqrt{\frac{W}{P}}$$

One of the questions considered at the Third International Road Congress (held in 1913) concerned the design and construction of water-bound macadam roads. Of eight reports dealing with this question presented at the meeting, that of Charles Lelievre [20] is most interesting and constructive. In reading the report one cannot help but be impressed with the fact that the author had a broad perspective of the problem in general and an appreciation of the need of designing roads according to scientific principles.

After discussing the function of the different component parts of the road structure he commented on the fact that there is an undeniable relationship between subgrade support and foundation and wearing course thickness that should be considered in the rational economic construction of a road. He called attention to the need for conducting experiments to determine the area over which traffic loads are distributed in road surfaces and for finding out what intensities of pressure are transmitted through the surface and foundation course to the subgrade. Lacking such data, by resorting to the use of information developed in 1877 by Leger in the course of a study of the distribution of forces in solid bodies conducted by means of polorized light, Lelievre attempted to obtain a solution to the problem of load distribution in road surfaces.

From his analysis he concluded that:

> When the upper surface of a road is pressed vertically by a force covering an area of diameter d the extreme lines of pressure in the interior of the road assume the form of a bell, having their apex within d and embracing at the lower base of the surface a zone of diameter D such that the difference $D-d$ does not exceed 2½ times the thickness of the surface.

For foundation course mixtures he decided for reasons not stated that the difference $D-d$ should not exceed 1½ times the thickness of the course.

Using these relationships, Lelievre made a series of calculations that served to indicate the intensity of pressure that would be apt to develop on the subgrade beneath road surfaces of different thicknesses. The results of his calculations, for an assumed load of 8,816 pounds resting upon the rim of a wheel 5.51 inches wide are given in table 2.

The relationships used by Lelievre may be expressed algebraically as follows:

For surface courses

$$P=\frac{W}{\pi(2.5T+d)^2/4}$$

For foundation courses

$$P=\frac{W}{\pi(1.5T+d)^2/4}$$

[20] Construction of Water-bound Macadamized Roads. Proceedings . . . International Road Congress. Report No. 81. 1913.

TABLE 2.—*Calculated intensity of pressure on the subgrade caused by a load of 8,816 pounds*

SURFACE COURSES

Thickness, inches	1.97	3.94	5.91	7.87	9.84	11.81
Subgrade pressure, pounds per square inch	102.5	47.7	27.4	17.4	12.3	9.1

FOUNDATION COURSES

Thickness, inches	5.91	7.87	9.84	11.81	15.74	39.37
Subgrade pressure, pounds per square inch	56.0	37.4	27.2	20.7	13.4	2.7

COMBINED SURFACE AND FOUNDATION COURSES

Thickness of foundation course, inches	5.91	7.87	9.84	11.81	27.56	39.37
Subgrade pressure, in pounds per square inch, for surface courses of:						
3.94 inches	19.3	14.7	12.6	10.2	3.5	2.0
5.91 inches	13.2	10.9	9.1	7.7	3.1	1.7
7.09 inches		9.1				
7.87 inches	9.7	8.2	6.8	6.1	2.8	1.6

In which P = the unit pressure on the subgrade.
W = the applied wheel load.
T = the thickness of surface.
d = diameter of the load contact area.

In using these equations for computing the subgrade pressure beneath a pavement consisting of a surface and foundation course, it is necessary first to solve for the value of D for the surface course ($D=2.5T+d$) and substitute this value of D for d in the second formula.

MIXING OIL-TREATED GRAVEL IN THE CONSTRUCTION OF A FLEXIBLE-TYPE ROAD SURFACE. THE RESISTANCE THAT THIS TYPE OF SURFACE OFFERS TO DEFORMATION UNDER LOAD DEPENDS ESSENTIALLY UPON THE DEGREE OF COHESION AND INTERNAL FRICTION IT POSSESSES.

DISCUSSION OF OTHER FORMULAS

With regard to the values given in the table, attention is called to the fact that they have been referred to and accepted in this country as data derived from actual tests. The facts of the situation may be summed up as follows: A Mr. Washington who attended the Third International Road Congress as a delegate from New York State, in reporting his trip to the State Highway Commissioner,[21] listed the French pressure values with the comment:

> Very interesting to us are French tests of the amount of pressure exerted through the road on the subsoil by a wheel load of 4 tons with a 5.5-inch tire.

The data were reproduced subsequently in Engineering and Contracting (vol. 42, December 16, 1914) with the statement that:

[21] Report of the State Commissioner of Highways (New York), 1913, vol. 2.

> The following notes on the transmission of pressure through macadam to the subgrade were made by W. de H. Washington in the last annual report of the New York Highway Commission * * *. French tests on the amount of pressure exerted through the road on the subgrade by a wheel load of 4 tons * * * gave the following results * * *.

Two years later the values appeared in Agg's 1916 edition of Construction of Roads and Pavements. In 1927 Harger and Bonney[22] proposed a method of design for flexible pavements. In a discussion of the method they not only call special attention to the fact that their formula for thickness is in essential agreement with the French test data of Lelievre, as reproduced by Agg, but they also show the comparison in graphical form.

The Harger and Bonney formula is predicated upon the Massachusetts rule of pressure distribution at an angle of 45°, although it is modified slightly so as to permit taking into consideration the width of tire through which the load may be applied to the road surface. Expressed in terms of thickness, it is as follows:

$$T=\sqrt{\frac{W}{3p}+\frac{a^2}{9}}-\frac{a}{3}$$

In which T = the thickness of surface.
W = the load.
p = the unit pressure on the subgrade.
a = width of tire.

B. E. Gray[23] has suggested a formula of the same general type for figuring the thickness of flexible pavements. It takes into consideration the area rather than the width of tire contact as follows:

$$T=0.564\sqrt{\frac{W}{p}}-\frac{d}{2}$$

In which T = thickness of surface.
W = the load.
p = the unit pressure on the subgrade.
d = diameter of equivalent area of tire contact.

The above formulas assume that all types of flexible pavements will transmit superimposed loads to the subgrade at a definite and constant angle with the horizontal and that the pressure transmitted to the subgrade is uniform over the area affected. In view of these assumptions it is not surprising that so little consideration has been given to their use as a means of determining the necessary thickness of surface on new construction work.

It is a matter of common knowledge that the resistance that a bituminous or soil-aggregate mixture offers to deformation under load depends essentially upon the degree of cohesion and internal friction it possesses. Certainly if variations in these properties affect resistance they in turn will affect load distribution since resistance and load distribution are closely related. Moreover, it is known that the intensity of the pressure imposed upon the subgrade will vary, depending upon such factors as the character of the soil material and the flexibility of the surface through which the loads are transmitted.

[22] Highway Engineers Handbook, Harger and Bonney, vol. 1, Fourth Edition, 1927.
[23] The Design and Construction of Bituminous Pavements. Report presented at the Annual Meeting of Highway Engineers and Commissioners of Michigan, Houghton, Mich., 1934.

THE WELL-DESIGNED FLEXIBLE-TYPE ROAD ADEQUATELY SERVES MODERN TRAFFIC. THE LOAD-CARRYING CAPACITY OF SUCH A ROAD DEPENDS UPON THE SUPPORTING POWER OF THE SUBGRADE AS WELL AS UPON THE THICKNESS OF THE SURFACE.

If a formula for surface thickness is founded upon a given angle of load distribution, then the size of the load contact area through which the load may be applied should necessarily be taken into account. For this reason, of the formulas cited that of Gray and those of Lelievre are the most logical. The Massachusetts formula assumes that the load is concentrated at a point; the Harger-Bonney formula assumes that it is concentrated on a line equal in length to the width of tire. The former assumption is, of course, entirely untenable whereas the latter might be a reasonable assumption for steel tires but it can scarcely be considered so for any modern tire equipment.

OTHER FORMULAS FOR COMPUTING FLEXIBLE PAVEMENT THICKNESS PRESENTED

Recently two other methods have been suggested for designing flexible pavements, one by Hawthorn and one by Housel. Although both are based upon certain theoretical conceptions of soil resistance and pressure distribution rather than upon new or original test data, in certain respects they are more complete and logical than those already enumerated.

Hawthorn's method [24] is predicated upon the assumptions that the wheel load is distributed to the subgrade through a truncated cone of the surface course and that the subgrade support under the base of the cone is uniform. Considering that the load is applied on the road surface through a circular contact area of radius a and equating the load to the subgrade support, assuming θ as the shearing or load-distributing angle, measured from the vertical, he obtained the formula

$$t=\frac{1}{\tan\theta}\sqrt{\frac{P}{\pi q}}-a$$

In which t = the thickness of surface.
P = the wheel load.
q = the unit subgrade support.

When θ is 45° this formula is exactly the same as that of Gray, namely:

$$t=0.564\sqrt{\frac{P}{q}}-a$$

In recognition of the possibility that the angle of load distribution may vary over rather wide limits, Hawthorn proposes a method for determining this angle experimentally for different types of surfacing. The method involves the measurement with soil pressure cells of the maximum value of q on the subgrade directly beneath a load applied on the surface and the substitution of this value in the above formula, solving for θ.

The use of any of the formulas enumerated for determining the thickness of flexible pavements requires, among other things, quantitative knowledge concerning the bearing capacity of the subgrade soil. With present methods of test the development of adequate knowledge of this character in the field is out of the question not only because of the time and expense involved but because of the physical impossibility of simulating with any degree of certainty the stress

[24] A Method of Designing Non-Rigid Highway Surfaces, by George Edward Hawthorn. Bulletin No. 83. University of Washington, Engineering Experiment Station, 1935.

reactions to which subgrades are subject under pavements of the flexible type.

Hawthorn offers an indirect method for developing quantitative values of soil bearing power. He developed an equation based upon the generally accepted theory that the resistance that a soil offers to displacement under sustained load depends upon its cohesion and internal friction. This equation also takes into account the added resistance to displacement that the weight of a road surface may afford. The principle of the method is much the same as that advanced by Hogentogler and Terzaghi in 1929, wherein the bearing capacity of a long narrow loaded strip was equated in terms of the two physical properties of the soil mentioned above and the secondary load adjacent to the bearing strip. It was necessary, of course, for Hawthorn to modify this method to the extent of considering tire contact areas rather than narrow loaded strips.

While this method of design might be said to be more complete than those previously described because ways and means are suggested for ascertaining the angle of load distribution through the surfacing material and the bearing capacity of the subgrade, there is considerable reason to question whether it is any more rational in principle.

In the first place, in view of what has been said about pressure distribution beneath bearing plates of different rigidity, it is extremely questionable whether the angle of load distribution can be determined within a reasonable degree of accuracy in the manner prescribed. According to Hawthorn's formula the thickness of surface varies inversely as the tangent of the angle of load distribution. This means that a change in the angle of 1 degree corresponds to a difference in road surface thickness of almost 3 percent. Emphasis is thus placed upon the importance of having or developing an accurate expression of the angle of load distribution if values of pavement thickness calculated by the prescribed method are to be considered more than merely indicative.

In the second place, as far as the method suggested for evaluating subgrade bearing capacity is concerned, there is some question as to whether present methods of test will furnish values of the angle of internal friction and unit cohesive force of different soils suitable for use in a formula such as the author has developed.

DEVELOPMENT OF THE MOST RECENT THICKNESS FORMULA OUTLINED

Housel's method [25] of flexible pavement design is premised upon the conceptions that (1) the amount of sustained pressure that a flexible road mat can distribute beyond the confines of a loaded area depends upon its ability to transmit shear on the lateral surface of the column beneath the loaded area, and (2) the maximum resistance that a subgrade material (cohesive soil) offers to displacement is a function of its shearing resistance.

Briefly the major steps in the mathematical development of his thickness formula are as follows:

1.—Equation (1) $p_z = p_0 - \frac{4m_1t}{b} + w_1t$

In which p_z = the unit pressure on the subgrade beneath the loaded area on the mat.

p_0 = the unit pressure applied on the surface.
m_1 = the unit shearing resistance of the mat.
t = the thickness of mat.
b = the diameter of the load contact area (considered as circular).
w_1 = the weight of the mat.

The quantity $\frac{4m_1t}{b}$ represents the pressure per unit of contact area that is transmitted outside the central pressure column by virtue of the resistance the surface mat offers to punching shear. This was obtained by considering that the unit pressure so transmitted is equal to the unit shearing resistance of the mat times the surface area of the central column divided by the cross sectional area of the column; i. e. $\frac{\pi bt m_1}{\pi b^2/4}$. The quantity w_1t represents the weight of the surface mat, which in the equation is taken as an additional source of subgrade pressure

Specifically, equation (1) is intended to evaluate the intensity of pressure concentration which may be transmitted to the subgrade in terms of (1) size of loaded area, (2) thickness of surface mat, (3) shearing resistance of the mat material, and (4) weight of the mat.

2.—Equation (4) $p_z = 4\ m_2 + w_1 t + \frac{2m_1t}{b}$

In which p_z = the total unit bearing capacity of the subgrade.
m_2 = the unit shearing resistance of the subgrade soil.
w_1 = weight of mat.
t = thickness of mat.
m_1 = the unit shearing resistance of mat.
b = the diameter of the load contact area on the mat.

The quantity $4m_2$ represents that part of the resistance that the subgrade soil offers to displacement as a result of its shearing resistance (p. 123, Housel's report). The quantity w_1t represents the additional resistance that the weight of the mat affords against lateral movement of the soil; and the quantity $\frac{2m_1t}{b}$ represents the shearing resistance that the mat offers against such movement. The latter quantity was developed considering b as the diameter of the load contact area, $3b$ the diameter of the subgrade pressure area and $(\pi b + 3\pi b)t$ as the total shearing surface of the mat tending to resist upward movement of the soil. Thus $(\pi b + 3\pi b)tm_1$ represents the total resisting force offered by the mat, which, when divided by the cross sectional area adjacent to the load contact area and within the outer circle of the subgrade pressure area, becomes $\frac{2m_1t}{b}$ or the downward-acting force of the mat per unit of area.

Equation (4) then is intended to evaluate the total unit bearing capacity p_z of the subgrade in terms of (1) the shearing resistance of the material m_2, (2) the weight of the mat w_1t and (3) the shearing resistance of the mat m_1.

3.—Equation (5) $\left(p_0 = 4m_2 + \frac{6m_1t}{b}\right)$, obtained by combining equations (1) and (4), expresses the unit bearing capacity of the pavement structure in terms of

[25] Design of Flexible Surfaces, by W. S. Housel. Proceedings of the Twenty-third Annual Highway Conference. University of Michigan, 1937.

all the resisting stresses; i. e., the shearing resistance of the surface mat and the shearing resistance of the subgrade soil, and in terms of the mat thickness and the diameter of the load contact area.

QUESTIONS RAISED RELATING TO THE ABOVE FORMULA

Among the questions that might be raised concerning the above mathematical treatment of the problem are:

1. Has proper consideration been accorded the condition at the plane of contact between the flexible surface and the supporting medium? Can, for example, the pressure on a cohesive subgrade beneath a loaded area on a flexible pavement be considered as uniform? According to the test data of Dr. Press (see p. 205), the pressure transmitted to a moist loam soil through a flexible bearing plate, uniformly loaded, varied to an appreciable degree.

2. Does the quantity $4m_2$ in equation (4), which is considered the sole source of resistance that the subgrade soil offers to displacement by virtue of its ability to resist shearing stresses, take into account the size of the pressure area? It appears that this quantity is a valid expression of resistance only in case the load is considered as being applied to the soil through a narrow strip, infinite in length.

3. In evaluating the resistance, $\left(\begin{matrix}2 & m_1 & t\\ & b & \end{matrix}\right)$, that a surface mat may offer against lateral or upward displacement of the subgrade adjacent to the loaded area, is it proper to consider this resistance at the outer zone of pressure influence as shearing resistance? It appears that it is a question here primarily of bending resistance.

4. Housel takes cognizance of the fact that for certain types of cohesive soil, settlement resulting from vertical compression may predominate and that as a result the surface mat may not function effectively as an added source of subgrade resistance. For this reason he reduces the quantity $\frac{6\ m_1\ t}{b}$ in equation (5) to $\frac{4\ m_1\ t}{b}$.

The question arises in this connection as to whether the same arbitrary allowance should be made for all types of cohesive soils.

Earlier in this report the statement was made that adequate information at present is lacking concerning the effect and intensity of the dynamic forces that vehicles impose upon pavements of the flexible type. It is largely because of this fact that the methods of design which have been developed to date are predicated upon static load considerations. In calculating the thickness of surface with them, some sort of an arbitrary safety factor is sometimes introduced to compensate for the possible dynamic or impact effects that moving vehicles may have upon the pavement.

For example, Harger and Bonney advocate an allowance of 50 percent over and above the permissible static wheel load in computing pavement thickness with their formula [22] [26]. Hawthorn, in addition to advocating the same allowance for impact, would base thickness upon values of subgrade resistance not greater than 50 percent of the ultimate strength. Both Gray and Housel contend that no allowance need be made for impact because the dynamic resistance of flexible pavements, i. e., the resistance that materials composing such pavements offer to deformation under quickly applied loads, may entirely outweigh any actual impact forces.

[26] Rational Road Design, by F. T. Sheets. Engineering News-Record, vol. 114, no. 3, 1935.

SUMMARY

This report has been devoted to a digest and discussion of published material pertinent to the design of flexible pavements. It was pointed out that while our knowledge of subgrades has progressed to the point where they can be prepared more or less according to scientific principles or in such manner that they will provide reasonably uniform and constant support to road surfaces, adequate information is lacking concerning their quantitative bearing capacity, at least that type of information suitable for use in design formulas.

The question of pressure distribution was dealt with at some length. It was brought out that the intensity of pressure transmitted to flexible pavement subgrades may depend to a large extent upon the degree of rigidity of the surface mat as well as upon the character of the subgrade soil. In view of this it does not appear that we should attempt to apply our knowledge or conceptions of rigid footing stress distribution in soils directly to the problem of flexible pavement design.

Considerable space was devoted to a review of the methods of design of flexible pavements that have been formulated to date. While these methods have contributed to our understanding of the factors involved in the problem and have served to stimulate an interest in the problem generally, they are, as was pointed out, premised more upon existing conceptions of soil resistance and pressure distribution than upon any great amount of new or pertinent data. The discussion serves to emphasize the need for the development of more comprehensive test data relating directly to the problems of load-supporting capacity and pressure distribution of flexible pavements resting upon different types of subgrade soil.

Finally, the question of vehicle loads in their relation to pavement surfaces of the nonrigid type was discussed and the need for more adequate information, particularly on dynamic effects, pointed out. It is apparent that before the required thickness of a pavement can be determined by any formula the question as to what is the critical wheel load must be answered.

Specifically, the major parts of the general problem that demand first attention seem to be:

1. A study of the load-supporting and pressure-distributing ability of typical surfaces of the non-rigid type as influenced by—

   a. The magnitude of the load.
   b. The area of load application and the distribution of pressure intensity over the area.
   c. The time duration of the load.
   d. The physical characteristics of the surface course and of the subgrade.

It is probable that with suitable testing equipment much information of value could be developed from specially constructed test sections. However, the development of satisfactory pressure-measuring equipment for tests such as these is not a simple matter and an important preliminary task is the development of the necessary testing equipment.

2. The development of data that indicate more directly the safe load-supporting value of soils when subject to forces and displacements such as obtain under road surfaces of the nonrigid type. Factors that probably exert an important influence are

a. The size of the area of applied pressure.
b. The rigidity of the surface through which the pressure is applied.
c. The effect of restraint to vertical movement around the area of applied pressure.
d. The physical characteristics of the subgrade material.

3. The determination of the relative effects of slowly applied and suddenly applied forces in order that the critical load for design purposes may be known.

It is apparent that the general problem is one of wide scope and great complexity. It is one of obvious importance. An early solution is not to be anticipated but great progress will undoubtedly be made toward one by the combined and coordinated efforts of the many research agencies now exploring this field.

THE ANGEL OF SHAVANO

The Angel of Shavano, illustrated on the cover of this issue, is an image created by converging snow-filled ravines and can be seen in early summer from U. S. Highway 650 near Salida, Colo. It rests on the dished east face of Shavano Peak, which is a 14,179-foot peak in the College Peaks Range high in the Colorado Rockies. As the snows melt away during early summer thaws, only the deeper drifts remain, creating the image of an angel with outspread wings.

PUBLICATION ON DESIGN OF CAMP STOVES AND FIREPLACES MADE AVAILABLE

A publication on camp stoves and fireplaces for recreational areas has been prepared by the Forest Service of the United States Department of Agriculture in an effort to bring together the best information available at the present time. The discussion applies primarily to the problems presented in the camps and picnic areas in the national forests; but it may apply equally well to many other types of recreational areas. It should be valuable to highway engineers who occasionally supervise the construction of recreational facilities in connection with roadside development.

The discussion ranges from the most informal picnic fireplaces and portable stoves to the most intricate masonry ovens for long-time camping. It covers automobile stoves, cooking standards, campfire circles, fireplaces with top grates or plates and with or without chimneys, combined fireplaces and stoves, multiple stoves, warming fires and shelters such as for mountaintops, and barbecue pits and ovens, as well as the lay-outs for camp units. There are right and wrong ways shown for building foundations, fireboxes, stonework, and chimneys. The materials discussed are iron, brick, concrete, stone, and sand. Special attention is given to fire hazards, fuel problems, and artistic design in fitting the fireplaces into the natural background. The book is profusely illustrated with 30 plates, each containing several sketches.

"Camp Stoves and Fireplaces" may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C., for $1.50 per copy (in buckram).

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF DECEMBER 31, 1937

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,122,820	$551,410	64.3	$2,088,341	$1,044,170	89.2	$2,936,320	$1,468,155	132.3	$4,783,445
Arizona	1,431,770	1,042,305	72.6	1,233,292	797,498	52.1	497,850	301,159	29.1	978,847
Arkansas	2,760,421	2,757,784	183.6	1,413,140	1,409,457	86.1	23,505	23,177		2,208,501
California	6,502,940	3,601,359	147.0	5,721,259	2,948,700	93.6	3,442,246	1,910,483	55.3	448,711
Colorado	2,783,235	1,554,449	113.2	867,332	474,504	22.6	1,260,741	685,250	48.8	1,666,886
Connecticut	783,198	389,196	9.3	231,520	108,123	.8				1,513,588
Delaware	393,079	196,494	14.2	141,431	70,672	6.7	219,285	107,624	13.3	1,118,222
Florida	301,082	149,953	8.2	2,716,172	1,358,086	65.2	862,070	431,035	13.6	2,478,109
Georgia	1,920,365	944,168	109.1	4,590,292	2,295,141	209.7	2,889,994	1,444,997	126.9	3,943,310
Idaho	2,385,991	1,420,624	183.9	877,807	523,929	67.5	422,853	238,838	13.6	789,066
Illinois	9,189,422	4,535,766	256.4	5,721,961	2,856,873	167.8	4,972,000	2,373,500	82.7	1,210,163
Indiana	5,474,706	2,719,168	129.4	2,897,715	1,448,741	88.8	1,568,123	764,749	35.9	1,717,700
Iowa	6,562,611	2,971,928	210.2	4,306,510	1,919,583	127.8	2,117,085	984,450	60.0	211,327
Kansas	3,388,496	1,666,284	176.7	3,203,115	1,601,458	96.4	1,873,773	936,750	107.9	3,123,950
Kentucky	2,096,616	998,308	64.1	3,453,236	1,726,618	88.7	1,363,804	681,902	80.2	2,278,170
Louisiana	382,218	186,475	9.3	6,555,335	1,430,435	45.2	5,547,243	892,045	23.0	1,688,236
Maine	1,898,980	949,490	52.6	1,871,216	935,608	42.2	588,410	294,200	13.6	102,890
Maryland	930,330	465,145	13.4	1,808,594	902,534	28.9	460,360	230,180	.5	1,496,949
Massachusetts	3,989,577	1,994,777	19.0	2,250,345	1,125,182	4.5	182,696	91,348	.5	1,877,025
Michigan	5,969,610	2,964,646	156.9	5,918,610	2,959,805	140.5	1,648,171	749,260	25.7	48,175
Minnesota	5,726,156	2,851,531	263.1	3,029,921	1,503,716	125.1	1,881,072	930,536	84.1	1,159,140
Mississippi	1,992,300	993,411	92.4	3,846,200	1,967,890	178.3	2,205,520	1,096,860	89.7	2,575,015
Missouri	7,988,021	3,990,319	431.5	4,544,605	2,176,287	126.1	3,437,741	1,351,861	74.3	1,321,959
Montana	3,884,103	2,177,243	283.7	1,867,927	1,050,333	86.4	571,734	321,594	33.8	1,969,638
Nebraska	2,365,131	1,182,565	239.8	4,414,743	2,195,350	409.2	2,772,648	755,215	92.7	1,998,636
Nevada	2,343,933	2,020,937	106.8	621,794	536,243	61.2	168,986	142,722	6.2	670,281
New Hampshire	360,696	177,963	6.5	518,529	256,321	9.5	29,819	14,909		985,626
New Jersey	1,674,607	760,009	20.2	1,180,410	589,885	6.5	1,921,170	959,750	12.7	1,171,595
New Mexico	3,010,136	1,844,775	219.0	2,177,374	1,427,886	109.8	230,230	130,729	6.9	243,974
New York	12,570,989	5,925,156	217.9	13,590,977	6,536,060	226.9	2,673,640	1,322,177	42.6	258,061
North Carolina	3,703,171	1,848,284	286.0	4,956,705	2,307,803	194.1	2,300,553	1,113,772	106.0	1,842,485
North Dakota	1,037,450	1,037,450	186.5	1,310,220	1,289,710	77.4	494,172	494,172	48.3	2,900,901
Ohio	3,486,353	1,672,663	47.4	9,209,649	4,570,066	100.5	1,735,295	868,083	20.6	4,993,796
Oklahoma	2,681,973	1,406,287	120.5	2,574,355	1,330,599	104.6	1,772,959	899,587	83.5	3,045,669
Oregon	3,616,277	2,138,059	126.9	1,521,346	927,815	64.3	863,711	488,280	35.5	729,582
Pennsylvania	11,070,339	5,506,738	155.6	6,741,086	3,351,103	92.1	3,557,147	1,777,536	64.2	2,238,829
Rhode Island	824,252	403,311	7.1	1,077,910	538,955	14.0	8,930	4,465		758,165
South Carolina	2,798,904	1,182,403	231.4	5,028,250	2,109,405	211.9	1,762,019	765,335	86.1	799,561
South Dakota	2,462,981	1,392,121	247.0	1,621,376	896,530	173.5	443,673	246,033	58.3	3,012,577
Tennessee	1,428,656	713,399	61.1	1,725,904	862,952	49.0	1,579,280	789,640	46.5	4,177,526
Texas	9,569,063	4,777,227	682.6	9,700,980	4,833,724	440.6	5,252,642	2,518,629	312.9	4,220,909
Utah	1,205,372	861,169	115.2	910,090	646,212	75.2	34,070	24,390	.5	1,080,148
Vermont	1,010,512	489,925	29.2	1,679,730	739,094	43.1	6,730	3,169		61,544
Virginia	2,627,752	1,313,876	124.1	2,587,202	1,247,525	65.3	3,026,407	1,509,504	89.5	870,232
Washington	1,820,861	944,478	73.1	3,249,031	1,704,312	39.9	560,860	294,700	7.9	550,674
West Virginia	901,295	455,248	24.3	1,576,571	787,379	42.8	635,792	514,831	17.5	1,935,697
Wisconsin	7,916,680	3,795,535	260.3	4,562,401	2,096,577	101.9	289,791	133,900	10.1	1,122,102
Wyoming	2,460,514	1,502,528	255.6	1,290,663	778,272	142.5	199,109	122,996	17.1	123,808
District of Columbia										
Hawaii	537,013	260,604	8.9	521,869	257,937	9.0	325,610	160,750	4.8	1,148,917
Puerto Rico				791,809	393,925	16.0	286,750	143,375	4.3	87,700
TOTALS	163,342,957	85,684,943	6,917.1	156,296,850	77,846,983	4,921.0	73,904,589	34,508,602	2,319.5	81,738,017

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF DECEMBER 31, 1937

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$4,151,115	$3,922,906	$3,874,971	136.9	$250,100	$243,478	7.8				$32,666
Arizona	2,569,841	3,156,644	2,505,062	193.7	38,548	38,548					26,230
Arkansas	3,352,061	3,149,006	3,121,666	350.5	181,216	180,523	9.8				49,872
California	7,747,928	7,954,424	7,541,007	263.9	145,356	145,356					61,566
Colorado	3,395,263	2,382,800	2,290,828	99.4	89,597	89,596	6.0	$8,200	$8,200		1,006,639
Connecticut	1,418,709	1,188,498	1,100,096	17.9	232,031	206,330	4.5	124,130	64,435	0.2	47,848
Delaware	900,310	861,217	835,485	62.6	22,350	22,850	4.0	26,712	26,712	4.4	15,263
Florida	2,597,144	2,603,553	2,529,126	99.1	38,957	38,957					29,961
Georgia	4,988,367	1,907,300	1,865,928	109.2	2,298,324	2,029,944	105.6	442,460	442,460	29.6	650,634
Idaho	2,222,747	2,273,614	2,167,698	185.9	33,341	33,341					21,707
Illinois	8,594,009	8,123,932	7,877,907	465.0	685,010	685,010	23.2				131,092
Indiana	4,941,255	5,169,692	4,845,920	238.4	49,000	49,000					46,335
Iowa	4,991,664	5,237,605	4,885,935	528.3	105,902	104,865	.3				864
Kansas	4,994,975	4,728,602	4,658,563	370.6	242,985	240,313	20.1	51,590	51,590	2.1	44,010
Kentucky	3,726,271	3,596,827	3,430,251	355.0	271,221	271,221	3.5				24,799
Louisiana	2,890,429	2,822,001	2,524,286	166.2	300,666	240,049	1.6	97,087	97,070	10.4	29,024
Maine	1,676,799	1,630,328	1,616,413	74.4	60,849	58,250	1.7				2,136
Maryland	1,750,738	773,435	766,648	27.2	467,536	467,536	10.0	168,198	134,366	4.3	382,189
Massachusetts	3,262,885	2,191,459	2,191,383	18.0	829,930	439,160	.4	1,136,726	568,363	.8	63,979
Michigan	6,301,414	6,573,146	5,950,255	288.6	284,921	284,921	3.4	230,438	63,746	.3	2,492
Minnesota	5,277,145	6,403,162	5,184,516	896.0	96,799	88,962	5.9				3,667
Mississippi	3,457,552	2,991,155	2,986,533	209.7	395,446	394,406	25.9	10,800	10,300	.6	65,313
Missouri	6,012,652	5,290,224	5,140,206	776.9	790,784	736,082	.7	34,391	32,294		104,070
Montana	3,676,416	3,597,353	3,535,614	200.7	95,385	95,385	.1	8,462	8,462		36,955
Nebraska	3,870,739	3,437,332	3,327,793	362.3	445,830	445,830	8.3	70,270	70,270	1.8	26,846
Nevada	2,243,074	2,301,828	2,195,115	110.0	84,970	38,146	1.7				9,813
New Hampshire	945,225	875,636	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,281,435	1,261,330	27.8	1,832,989	1,832,989	7.6	34,468	34,468		1,018
New Mexico	2,871,397	2,811,536	2,806,370	213.7	43,071	43,071		14,581	12,196		9,760
New York	11,046,377	10,741,839	10,276,419	170.0	316,300	276,300	1.9	68,000	68,000	.2	425,658
North Carolina	4,720,173	4,521,721	4,453,105	277.2	194,791	194,791	13.5	37,900	37,900	.5	34,377
North Dakota	2,867,245	2,491,800	2,448,886	378.4	107,799	107,799	1.2	286,285	286,285	36.0	24,275
Ohio	7,670,815	6,958,421	6,842,574	287.9	730,632	724,632	8.8	30,200	30,200	1.9	73,409
Oklahoma	4,580,670	4,542,968	4,310,604	400.5	251,970	251,970	7.9	8,800	5,000	.1	13,096
Oregon	3,038,642	3,216,548	2,950,184	164.6	45,580	45,580		11,846	11,846	1.0	31,032
Pennsylvania	9,347,797	6,543,808	6,233,396	253.7	3,084,682	2,676,346	28.3	368,646	273,316	3.9	164,739
Rhode Island	989,208	1,111,600	989,136	18.8							72
South Carolina	2,702,012	2,290,110	2,110,868	224.6	617,029	545,752	24.9	9,664	9,664	1.5	35,728
South Dakota	2,976,454	2,689,680	2,685,957	482.6	270,437	270,437	22.0				20,060
Tennessee	4,192,460	3,432,415	3,381,237	134.3	778,283	778,283	16.5	35,710	22,060	2.6	10,881
Texas	11,989,350	12,587,020	11,525,419	1,105.8	458,571	346,058	11.9	83,337	83,100	7.2	34,773
Utah	2,067,154	2,150,682	1,929,726	208.2	112,055	112,055	.1				25,372
Vermont	924,306	1,060,294	913,206	23.2	11,100	11,100					
Virginia	3,652,667	3,365,922	3,261,799	989.0	172,141	168,344	14.2				222,024
Washington	3,026,161	3,316,804	2,940,422	164.3	65,824	65,824	.7				19,915
West Virginia	2,231,412	1,976,515	1,880,973	81.0	375,022	317,639	14.5	32,300	32,300	4.9	
Wisconsin	4,823,884	5,231,164	4,729,389	343.4	94,268	93,900	.3				595
Wyoming	2,219,155	2,170,832	2,165,171	152.4	33,287	33,287					20,697
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	678,560	655,700	10.4	277,293	215,689	7.0	62,530	54,644	.6	
TOTALS	195,000,000	181,265,353	171,493,325	12,764.7	18,531,770	16,882,206	431.5	3,494,331	2,540,247	114.9	4,083,722

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF DECEMBER 31, 1937

STATE	APPORTIONMENT	COMPLETED		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$4,034,617	$3,554,270	$3,538,941	47	1	12	$378,519	$378,519	2			$175,300	$109,000	4			$8,157
Arizona	1,256,099	1,216,505	1,170,445	14		3	85,121	62,712	1		1	2,280	2,280			1	20,662
Arkansas	3,574,060	2,825,257	2,817,988	51	6	7	687,340	686,635	5		17	23,923	23,923			9	45,514
California	7,486,362	7,210,651	7,023,934	46	8		414,005	414,004	1			10,000	10,000	5			38,424
Colorado	2,631,567	2,262,520	2,193,165	27	1	8	437,794	437,794	3	2	13						608
Connecticut	1,712,684	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	130,000	1			279,052	279,052	2								9,187
Florida	2,827,883	2,408,845	2,381,150	30	5	2	119,018	119,018	1			136,010	136,010			74	191,705
Georgia	4,895,949	702,989	701,485	16	5	10	1,357,729	1,357,729	22	8	7	939,100	939,100	12	2	35	1,897,635
Idaho	1,674,479	1,392,801	1,366,439	20	3	12	252,696	252,696	2		6	4,261	4,261			3	51,083
Illinois	10,307,184	8,803,088	8,675,516	65	8		1,500,134	1,493,730	9			101,000	101,000	2			36,938
Indiana	5,111,096	4,370,132	4,248,088	39	13		857,447	857,447	3		162						5,561
Iowa	5,600,679	4,589,282	4,493,882	98	9	8	1,092,875	1,089,000	8								17,797
Kansas	5,246,258	3,951,273	3,929,165	54		5	1,273,119	1,253,704	4	1		115,024	22,020	2		3	41,369
Kentucky	3,672,387	1,359,956	1,341,033	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		24,288
Louisiana	3,213,467	1,466,944	1,466,936	15	1		982,285	982,265	10	1		649,599	600,110	4	1		164,155
Maine	1,426,861	1,266,773	1,263,012	19	1	4	160,788	142,540	1	1							21,309
Maryland	2,061,751	660,619	660,481	5	3	18	865,674	815,674	5	2	6	292,562	270,093	1	1	6	315,503
Massachusetts	4,210,833	2,605,349	2,602,919	21	4		1,189,410	1,189,410	5			249,991	249,991	1			168,513
Michigan	6,765,197	6,872,919	6,563,144	44	8		101,477	101,477				175,500	84,230	1	1		16,346
Minnesota	5,395,441	4,765,523	4,654,827	82	13	50	676,480	670,430	4								70,184
Mississippi	3,241,475	2,210,035	2,204,881	49	4	8	524,453	524,453	6	3	3	71,900	71,900	2		12	440,241
Missouri	6,142,153	4,276,951	4,106,511	39	1		2,029,178	2,029,178	10		1	1,650	1,650			1	4,814
Montana	2,722,327	2,580,711	2,530,870	37	7		245,576	185,931	1	1							5,526
Nebraska	3,556,441	2,896,387	2,852,853	76	3	21	470,813	470,813	5		1	199,683	199,683	3		2	33,093
Nevada	887,260	877,030	845,455	8	3		13,308	13,308				3,630	3,630			5	24,857
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,058,833	3,045,868	20	5	1	831,570	831,570	4	4		44,730	44,730		1		61,658
New Mexico	1,725,286	1,700,977	1,694,089	19	1		25,879	25,879									5,317
New York	13,577,139	10,893,641	10,607,445	34	42		2,683,270	2,683,270	11	7		94,000	94,000				192,474
North Carolina	4,823,958	3,519,973	3,499,838	48	15		1,199,009	1,199,009	14		115						125,110
North Dakota	3,207,473	2,817,411	2,813,288	56	4		389,012	389,012		1							5,173
Ohio	8,439,897	2,494,830	2,366,019	19	6	2	5,506,734	5,038,588	28	2	8	843,940	828,940	12		4	206,351
Oklahoma	5,004,711	3,667,115	3,653,002	57	9	13	1,353,584	1,220,484	8	3	8	64,150	64,150			20	67,075
Oregon	2,334,204	2,329,684	2,242,111	16	6	2	74,215	74,215	1								17,878
Pennsylvania	11,483,613	7,730,138	7,217,210	65	18	9	4,195,716	3,828,728	21	5		150,473	150,473	5			287,202
Rhode Island	699,691	701,008	695,039	4	3												4,652
South Carolina	3,059,956	1,662,668	1,642,405	32	11	25	916,922	909,827	14	4	11	176,367	176,367	1	1	12	331,358
South Dakota	3,249,086	2,358,402	2,857,001	59	6	53	175,186	175,186	6			269,350	210,393	17			6,506
Tennessee	3,903,979	1,475,878	1,467,197	27	3	29	2,070,150	2,070,150	18	1	1	130,360	130,360	1		10	236,272
Texas	10,855,982	9,807,750	9,795,560	125	14	82	555,744	555,744	3	1	13	167,019	167,019			54	337,659
Utah	1,230,763	1,203,470	1,196,574	17	1	1	18,461	18,461									15,728
Vermont	729,357	748,907	703,707	10	7	18	14,100	14,100			2						12,050
Virginia	3,774,287	2,742,102	2,605,579	43	16	21	1,117,828	1,111,203	9	4	1	33,140	33,140		1	2	24,365
Washington	3,095,041	2,719,915	2,696,744	22	11	11	368,494	368,069	1	1							30,228
West Virginia	2,677,337	1,071,821	1,069,853	10	2	5	1,525,782	1,525,281	16	1	1	78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,558,834	4,522,667	37	6	2	398,588	398,385		1	11	105,746	38,299			21	63,332
Wyoming	1,360,841	1,220,946	1,203,943	13			111,212	111,212	1								45,681
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			226,162	169,698	2								
TOTALS	196,000,000	146,214,623	143,323,744	1673	306	449	42,946,496	41,384,957	280	60	388	5,924,228	5,460,292	78	9	276	5,831,007

U. S. GOVERNMENT PRINTING OFFICE: 1938

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

DEPARTMENT BULLETINS

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TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

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Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF DECEMBER 31, 1937

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMED PROJECTS	
	Sec 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 15,820,902	$ 8,310,690	$ 3,884,269	772.7	$ 72,922	$ 55,922	$ 17,000	4.0		$ 285,822	1.5	$ 3,521	$ 72,751
Arizona	5,211,960	2,641,935	9,036,132	5,211,960	2,616,561	537.0	19,573		19,573						5,801
Arkansas	6,748,335	3,428,049	11,131,031	6,747,820	3,412,141	637.3	11,500		11,500			3,000		515	1,408
California	15,607,354	7,932,206	30,610,401	15,607,354	7,796,202	763.9	114,585		114,383						21,621
Colorado	6,874,530	3,486,006	11,268,935	6,874,530	3,463,635	633.2	11,000		11,000						11,371
Connecticut	2,865,740	1,454,868	4,621,972	2,814,446	1,390,242	74.4	119,440	47,854	60,759	1.1				3,440	3,866
Delaware	1,819,088	923,395	2,791,041	1,819,088	923,395	128.9									
Florida	5,231,834	2,661,343	9,164,769	5,175,534	2,529,961	309.6	174,220	56,300	117,920	.5					13,462
Georgia	10,091,185	5,113,491	14,386,733	9,537,910	3,906,027	828.3	667,153	179,131	488,022	13.7	$ 312,513	461,598	10.6	61,632	257,844
Idaho	4,486,249	2,277,486	7,184,172	4,486,249	2,243,324	503.4	34,162		34,162						
Illinois	17,570,770	8,921,401	26,903,647	17,504,187	8,095,985	750.8	819,204		784,944	4.8		2,400	.8	66,583	38,072
Indiana	10,037,843	5,088,963	15,816,085	10,017,034	4,902,607	485.1	3,000		3,000			90,228	.9	20,809	93,129
Iowa	10,055,660	5,118,361	15,797,954	10,055,660	5,118,361	1,227.3									
Kansas	10,089,604	5,117,675	15,539,816	10,088,842	5,096,436	1,145.4						20,885	.2	761	354
Kentucky	7,517,359	3,818,311	12,227,767	7,480,082	3,769,850	814.7	32,565		32,565	.8	11,306	3,982		25,971	11,915
Louisiana	5,828,591	2,963,932	9,466,082	5,776,336	2,837,706	272.2	83,804		83,799	5.1				52,255	42,427
Maine	3,369,917	1,711,586	5,328,990	3,363,367	1,702,679	195.0						8,644		6,550	262
Maryland	3,564,527	1,810,058	5,735,982	3,471,009	1,111,390	153.7	547,475	59,747	482,208	7.4	33,770	103,930	1.8		112,530
Massachusetts	6,597,100	3,350,474	10,005,946	6,550,778	2,686,717	115.4	492,880		492,880	.1		91,944	.1	46,322	78,934
Michigan	12,736,227	6,452,568	20,880,502	12,736,227	6,355,780	768.5	96,788		96,788						
Minnesota	10,656,569	5,425,551	16,359,188	10,559,762	4,957,493	1,648.0	534,741	67,612	375,923	2.0				29,195	92,135
Mississippi	6,978,675	3,540,227	13,123,933	6,780,814	3,312,455	725.2	353,584	162,667	188,783	19.5				35,194	38,988
Missouri	12,180,306	6,173,740	18,990,709	12,116,921	5,837,130	1,447.3	283,560		283,560	.3				63,385	53,050
Montana	7,439,748	3,769,734	11,760,785	7,404,944	3,655,149	1,058.4	128,470	34,791	93,679	.2				12	20,906
Nebraska	7,828,961	3,964,364	13,182,736	7,828,961	3,917,425	1,055.2	39,755		39,755	.4					7,184
Nevada	4,545,917	2,302,356	7,074,562	4,545,917	2,265,770	758.8	21,979		21,979			11,302	1.7		3,305
New Hampshire	1,909,839	969,462	3,007,927	1,904,951	955,186	78.3					4,889	14,276			
New Jersey	6,346,039	3,220,879	10,314,432	6,265,478	3,074,353	97.0					58,170	47,500		22,391	98,426
New Mexico	5,792,935	2,941,700	8,909,776	5,748,150	2,917,944	750.0	55,585	44,785	10,800	1.2		2,695			10,261
New York	22,330,101	11,327,921	40,575,804	22,224,878	11,002,926	825.1	192,932	6,831	59,900	.1	27,881	115,400	.2	70,511	149,695
North Carolina	9,522,293	4,840,941	15,506,637	9,508,881	4,764,367	1,398.1	67,907		67,907	.3	5,960	8,200		7,452	466
North Dakota	5,804,448	2,938,967	9,070,516	5,729,310	2,545,435	2,170.3	1,681		1,681			332,106	15.6	75,138	59,745
Ohio	15,484,592	7,865,012	25,093,803	15,480,362	7,667,565	800.3	117,975		117,975					4,230	79,472
Oklahoma	9,216,798	4,685,180	14,788,739	9,214,165	4,454,062	806.6	227,954		227,954	.4				2,633	3,164
Oregon	6,106,896	3,097,814	10,051,231	6,101,165	2,939,201	469.4	46,467		46,467			35,000	.2	5,731	17,146
Pennsylvania	18,891,004	9,590,788	30,053,597	18,728,767	9,275,971	1,063.8	369,647	102,810	223,581	7.0				59,427	91,236
Rhode Island	1,998,708	1,014,572	3,150,270	1,998,708	1,012,094	89.1	2,478		2,478						
South Carolina	5,459,165	2,770,954	8,332,223	5,367,552	2,602,838	634.3	213,632	60,363	145,536	3.0				31,250	22,581
South Dakota	6,011,479	3,047,643	9,533,537	5,984,519	2,965,938	1,622.3	73,081		68,261	6.0	25,810	6,540	13.3	1,150	6,904
Tennessee	8,492,619	4,302,991	13,922,660	8,492,551	4,300,325	504.3								68	2,666
Texas	24,244,024	12,291,253	38,027,636	24,192,596	11,899,635	2,782.1	358,592	2,797	355,794	.4	40,268	1,952	1.0	8,363	33,872
Utah	4,194,708	2,132,691	7,464,635	4,194,708	2,132,691	590.9									
Vermont	1,867,573	948,007	3,181,483	1,867,573	948,007	141.0									
Virginia	7,416,757	3,765,387	11,951,109	7,366,684	3,685,487	671.6	109,038	18,439	51,176	2.4	1,500	10,000		30,134	18,725
Washington	6,115,867	3,106,412	9,408,337	6,112,042	3,018,659	303.0	85,353	3,825	74,069	1.2		5,192	.3		8,491
West Virginia	4,474,234	2,280,335	6,695,731	4,308,274	2,014,748	216.4	150,851	54,892	95,960	5.7		154,939	1.6	111,069	14,688
Wisconsin	9,724,881	4,941,837	15,457,804	9,724,881	4,871,807	619.6	40,860		40,860			14,200			14,970
Wyoming	4,501,327	2,287,712	6,954,439	4,500,651	2,244,357	1,052.2	17,900		17,900	.6		570		676	24,885
District of Columbia	1,918,469	973,842	2,892,447	1,918,469	973,842	22.3									
Hawaii	1,871,062	949,778	3,338,323	1,857,812	904,994	55.9	14,000		14,000		13,250				30,784
TOTALS	394,000,000	200,000,000	641,893,868	391,659,549	191,021,722	35,573.6	6,808,293	958,766	5,476,481	88.2	535,317	1,832,305	49.8	846,368	1,669,492

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 18, No. 12 February 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE STRUCTURAL DESIGN OF FLEXIBLE PIPE CULVERTS[a]

Reported by M. G. SPANGLER, Associate Structural Engineer, Iowa Engineering Experiment Station

CORRUGATED METAL PIPES have been utilized in the construction of culverts and other underground conduits for a period of about 40 years. Their use in highway and railway culvert construction has grown from a modest beginning to a place of major importance as a cross-drainage structure. The relatively light weight of this type of culvert pipe, with the attendant advantages in ease of transportation and placement, has been a powerful factor in the highly competitive field of culvert pipe manufacture and installation.

In spite of this long record of use, however, there is no rational method available to the engineering profession for predicting the structural performance of this flexible type of culvert pipe before installation, and there is need for a valid design procedure based upon recognized principles of mechanics.

This paper is a progress report on a research project being conducted cooperatively by the Iowa Engineering Experiment Station and the United States Bureau of Public Roads for the purpose of (1) supplying information relative to the structural performance of flexible pipe culverts under earth embankments, and (2) developing a rational theory of design for this type of structure.

A hypothesis of the magnitude and distribution of the various forces to which a flexible pipe culvert is subjected when installed as a projecting conduit without struts or other prestressing devices, and of the behavior of the structure in response to these forces, will be offered. A description of some extensive experimental work that has been conducted for the purpose of verifying the hypothesis will also be given, together with a comparison between the actual and hypothetical performance of the experimental culverts. The paper will deal wholly with structural aspects of corrugated metal pipe culverts and will not go into the matter of durability of the various metals of which they are made.

Culvert pipes placed under earth embankments derive their ability to support the earth above them from two sources: First the inherent strength of the pipe to resist external pressures; and second, the lateral pressure of the earth at the sides of the pipe, which produces stresses in the pipe ring in opposite directions to those produced by the vertical loads and therefore assists the pipe in supporting the vertical loads. In rigid pipes such as those made of concrete, cast iron, burned clay, etc., the inherent strength of the pipe is the predominant source of supporting ability. The only lateral pressure that can be safely depended upon to augment the load-carrying capacity of the pipe is the active lateral pressure of the earth, since the rigid pipes deform very little under the vertical load and consequently the sides do not move outward enough to develop any appreciable passive pressure in the enveloping earth.

In flexible pipes, however, the situation is reversed. The pipe itself has relatively little inherent strength, and a large part of its ability to support vertical load must be derived from the passive pressures induced as the sides move outward against the earth. The ability of a flexible pipe to deform readily and thus utilize the passive pressure of the earth on each side of the pipe is its principal distinguishing structural characteristic and accounts for the fact that such a relatively light-weight, low-strength pipe can support earth fills of considerable height without showing evidence of structural distress. It is apparent from these considerations that any attempt to analyze the structural behavior of this type of culvert pipe under a fill must take into account the earth at the sides as an integral part of the structure, since so much of the total supporting strength depends upon the side material.

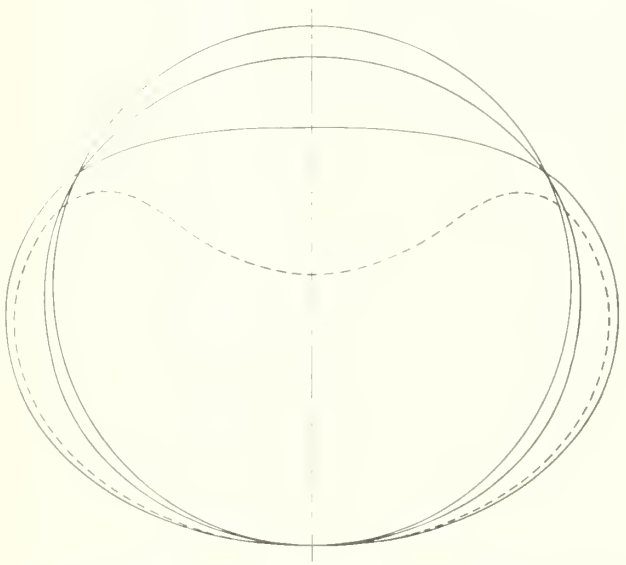

FIGURE 1.—SHAPES TAKEN BY A FLEXIBLE CULVERT PIPE DURING DEFLECTION UNDER VERTICAL LOAD.

The normal sequence of deflection of a flexible pipe culvert installed in the ordinary manner under an earth embankment that is built high enough to cause the pipe to collapse may be summarized as follows: The first layers of the fill cause the pipe to deflect appreciable amounts, the vertical diameter becoming less and the horizontal diameter greater. The outward movement of the sides of the pipe against the enveloping earth brings into play the passive resistance of the earth, which acts horizontally against the pipe and materially reduces the amount that the ring would deflect if acted upon by the vertical earth loads alone.

This action continues as the embankment is built higher until the top of the pipe becomes approximately flat, when additional load will cause the curvature of the top portion of the pipe to reverse direction, becoming concave upward (see fig. 1). The sides of the pipe will then pull inward. This eliminates the side supports of the pipe, since they are passive forces that cannot follow the inward movement, and the pipe will proceed rapidly to collapse and fail completely. The whole action is one of deflection change unaccompanied by rupture or buckling of the metal ring, although the material in certain parts of the ring may be stressed well beyond its elastic limit during the process.

[a] Paper presented at the Seventeenth Annual Meeting of the Highway Research Board, Washington, D. C., 1937.

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It seems evident, therefore, that any attempt to rationalize the structural design of flexible pipe culverts should be directed toward the development of a method for predetermining the deflection of the pipe under specified conditions of installation.

ELASTIC THEORY AS APPLIED TO THIN RINGS OUTLINED

In the elastic theory of flexure as applied to thin rings it is shown that

$$\frac{1}{\rho}-\frac{1}{r}=\frac{M}{EI} \quad (1)$$

where

ρ=the radius of curvature of the elastic curve of the ring when loaded.
r=the mean radius of the unloaded ring.
$\frac{1}{\rho}-\frac{1}{r}$=the change in curvature of the elastic curve, caused by the load.
M=the bending moment at any point on the ring.
E=modulus of elasticity of the material.
I=moment of inertia of the ring cross section.

Also, by calculus it is known that

$$\frac{1}{\rho}-\frac{1}{r}=\frac{d\theta}{ds} \quad (2)$$

where $d\theta$ is the angle through which a plane section normal to the elastic curve rotates with respect to its original position, within a length of arc ds.

Then

$$\frac{d\theta}{ds}=\frac{M}{EI} \quad (3)$$

But $ds=\rho d\phi$, and it is assumed that the difference between ρ and r will be small so that likewise $ds=rd\phi$.[1]

Therefore

$$\frac{rM}{EI}=\frac{d\theta}{d\phi} \quad (4)$$

In this expression, $\frac{d\theta}{d\phi}$ represents the rate of change of the angular displacement of a normal section with respect to the central angle ϕ.

When the external loads on the ring are symmetrical about the vertical axis, the normal sections at the top and bottom of the ring will remain vertical regardless of the character and magnitude of the angular displacements at intermediate points, and the sum of all the elementary displacements as ϕ varies from 0 to π will be

$$\frac{r}{EI}\int_0^{\pi} Md\phi=0 \quad (5)$$

or, more simply

$$\int_0^{\pi} Md\phi=0 \quad (6)$$

From this equation, the moment and thrust caused by any system of loads that is symmetrical about the vertical axis [2] can be obtained for either the top or bottom point of the ring. Having the moment and thrust at either of these points, the moment, tangential thrust, and radial shear can be obtained for any point on the half circle by the equations of equilibrium. Likewise, the vertical and horizontal deflections of the pipe ring can be derived from this basic relationship.

[1] This is an assumption employed generally in thin ring analysis. Its applicability to flexible rings is shown by laboratory experiments described later in this paper.

[2] If the loads are unsymmetrical about the vertical axis, the equation is integrated between the limits 0 and 2π.

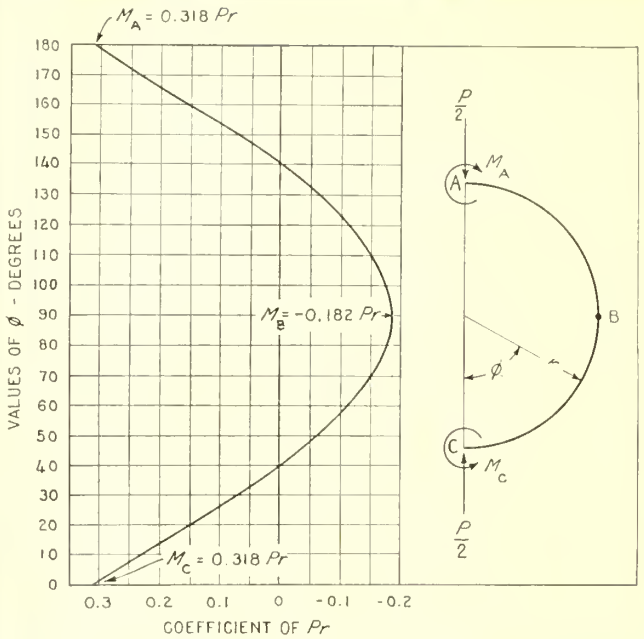

FIGURE 2.—VARIATION IN MOMENT AROUND A PIPE RING RESULTING FROM LOADS AT DIAMETRICALLY OPPOSITE POINTS.

For loads applied at diametrically opposite points on the ring, the expression for moment at the top and bottom is

$$M_A=M_C=0.318\ Pr \quad (7)$$

and at the sides

$$M_B=0.182\ Pr \quad (8)$$

in which

P=the load.
r=mean radius of pipe.

The variation in moment around the pipe ring is shown in figure 2. The expressions for deflections caused by this loading are:

For vertical deflection,

$$\Delta y=0.149\frac{Pr^3}{EI} \quad (9)$$

For horizontal deflection,

$$\Delta x=0.136\frac{Pr^3}{EI} \quad (10)$$

It will be noted that in the application of the elastic theory to a closed ring, the assumption is made that the radius of curvature of the loaded ring is nearly equal to the radius of the unloaded ring and r is substituted for ρ at one step in the analysis. For rigid rings, this assumption is easily accepted, but for flexible rings such as corrugated metal pipes, where the deflections and changes in the radius of curvature are relatively large, there was considerable doubt whether this assumption is valid. In order to obtain evidence on this point, some flexible pipe sections of various diameters were loaded in the laboratory at diametrically opposite points and the measured deflections were compared with those calculated by formulas (9) and (10). The physical properties of the pipe specimens were determined as outlined below.

During the laboratory loading experiments, the unit strain of the inner fiber of the pipe wall was measured at the top directly under the applied load by means of a Huggenberger tensometer having a gage length of ½ inch. The measured strains are shown in the load-strain diagrams in figures 3 to 6. The modulus of elasticity of the metal in each pipe was calculated by dividing

these measured strains into the unit stress obtained by the moment formula (7) and the flexure formula $\frac{Mc}{I}$. Thus

$$E=\frac{s}{\delta}=\frac{0.318\ Prc}{\delta I} \quad \text{(11)}$$

in which

E=modulus of elasticity, pounds per square inch.
δ=unit strain, inches per inch.
P=load per inch length of pipe.
r=mean radius of pipe, inches.
c=distance from neutral axis of the pipe wall to the outer fiber, inches. For standard corrugated sheets with corrugations ½ inch deep and 2⅔-inch pitch, $c=0.25+\frac{t}{2}$, where t=the thickness of the metal.
I=moment of inertia per inch length of pipe.

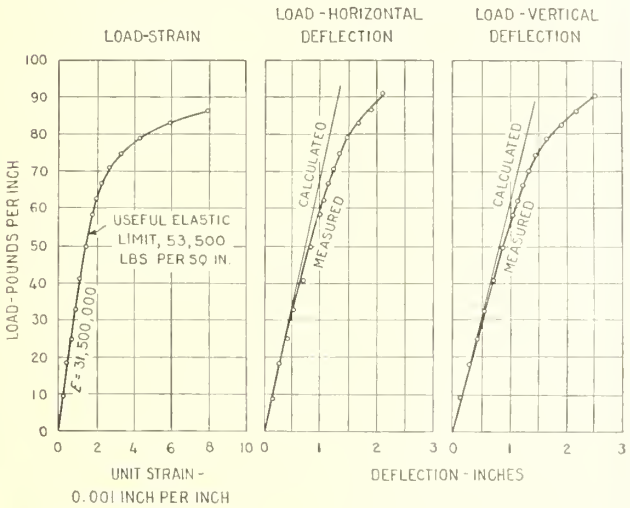

FIGURE 3.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 36-INCH PIPE.

CULVERT PIPE SPECIMENS TESTED IN THE LABORATORY

The moduli of elasticity of the metal in four specimens tested in the laboratory, as determined from measured strains by means of formula (11), were as follows:

Nominal pipe diameter (inches):	*Modulus of elasticity, lb. per sq. in.*
36	31, 500, 000
42	33, 200, 000
48	32, 000, 000
60	26, 800, 000

Each of these laboratory specimens was made from sheets rolled from the same heat of metal as were the test specimens of the same size used in the field-laboratory experiments to be described later, and these values of the modulus of elasticity may be used in analyses of the performance of the field test specimens.

The mean diameter of 8 test specimens of each size averaged approximately 1 inch greater than the nominal diameter and this mean diameter has been used in calculating the deflections of the pipes.

The value of the moment of inertia of the cross section of a standard corrugated sheet having corrugations ½ inch deep and spaced 2⅔ inches center-to-center has been determined by means of a formula developed several years ago by E. T. Jensen, when employed by

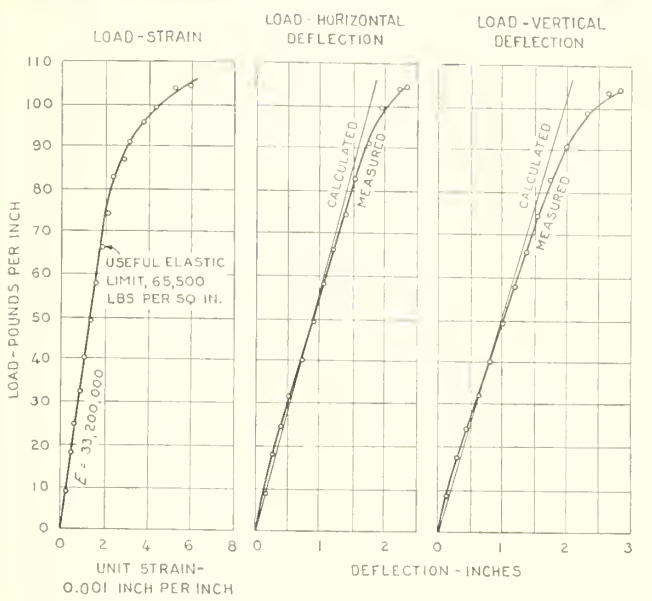

FIGURE 4.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 42-INCH PIPE.

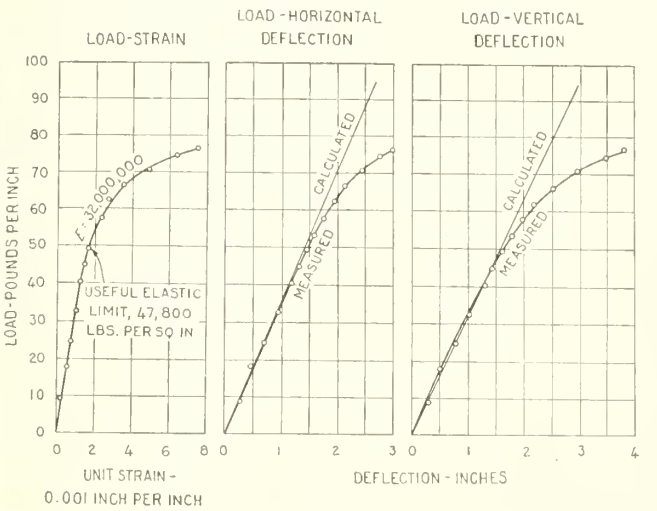

FIGURE 5.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 48-INCH PIPE.

the Iowa Engineering Experiment Station, as follows:

$$I=0.02925t-0.00150t^2+0.10425t^3-0.00225t^4 \quad \text{(12)}$$

G. E. Shafer has devised a simpler straight-line formula for moment of inertia which is

$$I=\frac{t}{30} \quad \text{(13)}$$

This formula gives moments of inertia that are up to 10 percent higher than values given by formula (12) within the range of thicknesses ordinarily encountered in corrugated metal culverts. Values of the moment of inertia and the section modulus for various gages by these two formulas are shown in table 1.

A graphical comparison between the two formulas for moment of inertia is shown in figure 7. The plotted points in this diagram are experimental moments of inertia obtained by Shafer by loading corrugated

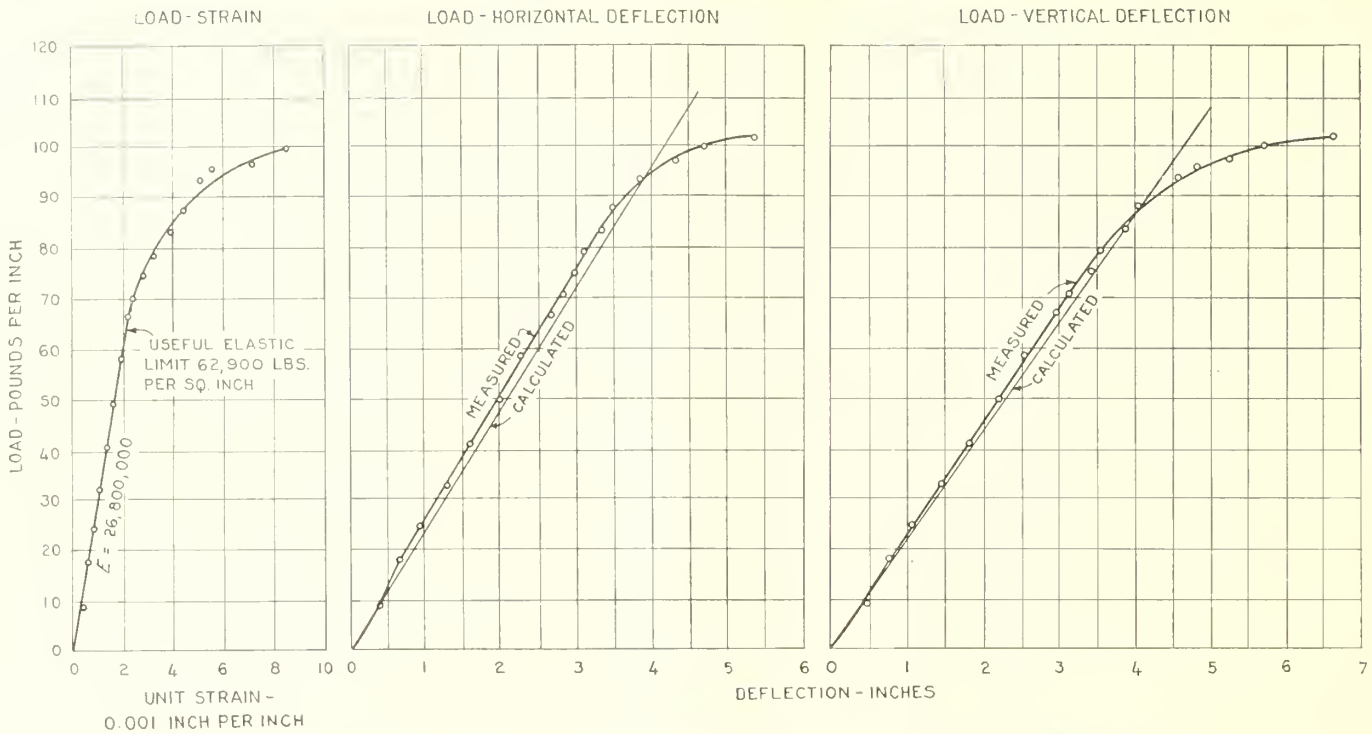

FIGURE 6.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 60-INCH PIPE.

sheets 48 inches long and 8 inches (three corrugations) wide as a beam with concentrated loads at the third points, and calculating I from observed deflections.

TABLE 1.—*Values of moment of inertia and section modulus for various gages of pipes* [1] *by two formulas*

U. S. Gage No.	Thickness	Jensen formula		Shafer formula	
		Moment of inertia, I	Section modulus	Moment of inertia, I	Section modulus
	Inch	*Inch*^{4}	*Inch*^{3}	*Inch*^{4}	*Inch*^{3}
4	0.234375	0.008275	0.022536	0.007813	0.021277
6	.203175	.006744	.019183	.006773	.019263
8	.171875	.005512	.016407	.005729	.017054
10	.140625	.004373	.013651	.004688	.014634
12	.109375	.003317	.010887	.003646	.011966
14	.078125	.002326	.008046	.002604	.009009
16	.062500	.001848	.006570	.002083	.007407
20	.037500	.001104	.004109	.001250	.004651
24	.025000	.000733	.002792	.000833	.003175
30	.012500	.000366	.001428	.000417	.002016

[1] Pipes have standard corrugations ½-inch deep and spaced 2⅔-inches center-to-center.

The measured deflections and those calculated by the thin-ring elastic theory as outlined above are close enough together (figs. 3 to 6) to justify the conclusion that this theory is applicable in the case of corrugated metal pipes under two-point loading. Even though the deflections and accompanying changes in radius of curvature are relatively large, the tolerance is probably no greater than that occasioned by variations in the modulus of elasticity of the metal, variations in thickness, depth, and spacing of corrugations, and other variables inherent in the manufacture of this type of conduit.

It seems tenable, therefore, to assume that the theory will also apply in the case of a corrugated culvert pipe installed under an embankment, since the external pressures on the pipe in the field will be more nearly uniformly distributed around the pipe than they were in the laboratory tests. An investigation of the structural performance of corrugated metal pipe culverts, therefore, becomes mainly a study of the magnitude and distribution of the loads and pressures to which they are subjected in service.

Research has been conducted in the general field of structural analysis and supporting strength of underground conduits of many types during the past 30 years, and some general conclusions have been reached relative to the behavior of such structures and the earth around them. One of these conclusions is that the earth embankment over a projecting conduit [3] produces vertical load on the structure in accordance with Marston's conduit load theory and this load is approximately uniformly distributed over the entire width of the conduit. This seems to be true regardless of the shape of the conduit. Also, in the case of circular pipes the vertical reaction is distributed over some narrower width, depending upon the width of bedding in which the pipe is laid and upon the rigidity or flexibility of the pipe.

In the case of rigid pipes the reaction, which is a passive pressure, will tend to be greater near the center of the pipe because of the circular shape. In the case of flexible pipes this tendency is counteracted partially by the large deflection of the pipe which flattens the bottom segment somewhat, thereby increasing the reaction near the edges of the bedding and bringing about a more nearly uniform distribution of pressure over the width of the bedding.

THEORY OF PRESSURE DISTRIBUTION AROUND FLEXIBLE PIPE CULVERTS DISCUSSED

Little is known concerning the magnitude of the passive resistance pressures that are developed at the sides of a flexible pipe as the pipe deflects and moves outward

[3] A projecting conduit is one that projects above the subgrade on which it is constructed, as opposed to a ditch conduit which is laid in a trench. A projecting conduit, therefore, is laid with its top some distance above the natural ground line, whereas the top of a ditch conduit is below the natural ground line.

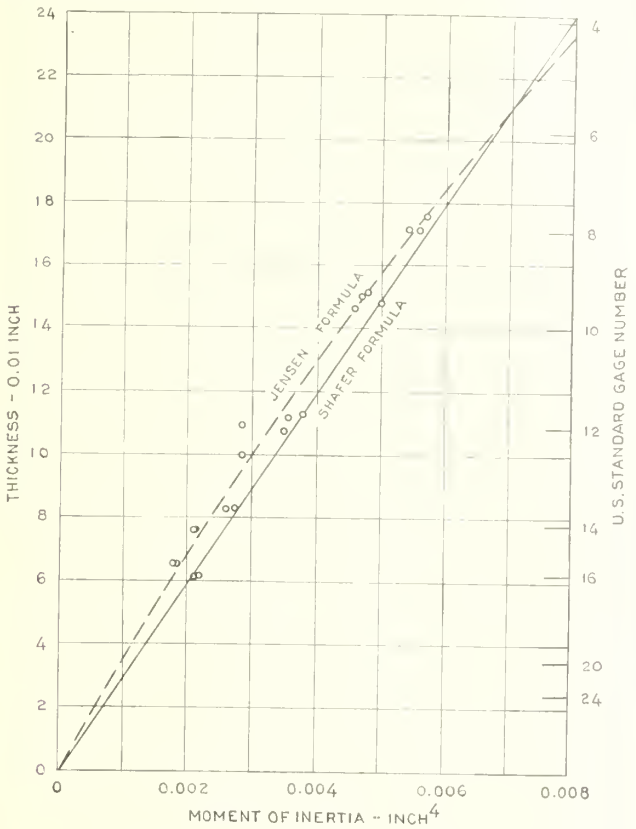

FIGURE 7.—COMPARISON OF MOMENTS OF INERTIA FOR VARIOUS GAGES OF PIPE BY TWO FORMULAS.

against the enveloping earth. In the Rankine theory of lateral earth pressures, the limiting value of the ratio of passive horizontal pressure to vertical pressure that a granular soil without cohesion can develop is shown to be the reciprocal of the active pressure ratio. Since active pressure, according to this theory, is roughly about one-third the vertical pressure, the passive pressure would be three times the vertical pressure or nine times the active lateral pressure. This theory does not give a clue as to the amount of movement required to develop the full limit of passive pressure, however.

At best Rankine's ratio would appear to establish a limiting value of passive pressure for a cohesionless soil, and is of little service when one attempts to assign specific values of pressure to definite movements or pipe deflections. Further, it is evident from the limited amount of work that has been done in this field that the relationship between passive pressure and movement is closely linked with soil characteristics and these characteristics will need to be carefully correlated with movement and vertical soil pressures in any rational treatment of flexible pipe design.

In two preliminary experiments, one begun in 1927 and the other in 1934, in which the structural action of two 42-inch corrugated pipe culverts was observed, measurements were made of the horizontal earth pressure at the ends of the horizontal diameter after the completion of each 1-foot increment of fill over the top of the pipes. Measurements of the horizontal diameter change for each increment of fill were also made. It was found that the ratio of horizontal pressure to diameter change was practically a constant regardless of the height of fill, in both these trials. Also, the values of this ratio were considerably different for the two types of soil of which the embankments were constructed, being approximately three times as great in the case of a gravel fill as it was for one built of loam.

Since the change in the horizontal diameter of a pipe is equal to the sum of the outward movements of the sides of the pipe at the two ends of the horizontal diameter, these preliminary observations have led to the tentative conclusion that, as a fill is built over a flexible culvert, the horizontal pressure on the pipe at any point bears a nearly constant relationship to the horizontal movement of the point. This constant ratio has been called the "modulus of passive resistance" of the fill material and is expressed as units of pressure per unit of movement such as pounds-per-square-inch per inch. Therefore, if the lateral movements of various points on the pipe ring are known, the distribution of lateral pressures on the pipe can be determined by multiplying the movement at each point by this modulus.

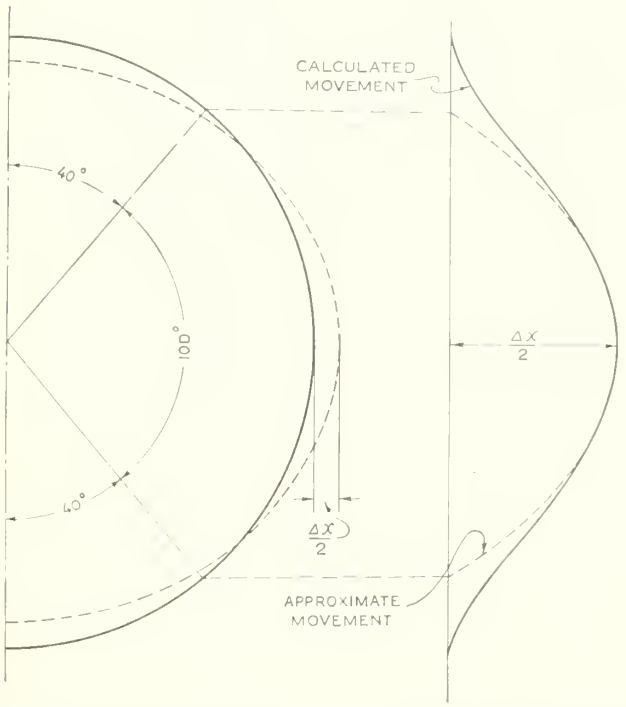

FIGURE 8.—CHANGE IN SHAPE OF A FLEXIBLE CULVERT PIPE UNDER LOAD. LEFT, CHANGE FROM CIRCLE TO ELLIPSE; RIGHT, AMOUNT OF HORIZONTAL MOVEMENT OF THE PIPE WHEN DEFORMED INTO AN ELLIPSE.

In general, it may be said that a corrugated metal culvert pipe under an embankment load deforms from a circular to an elliptical shape, with the minor (vertical) axis of the ellipse less than the diameter of the circle by the amount of the vertical deflection and the major (horizontal) axis greater than the diameter by the amount of the horizontal deflection. Also, for moderate deflections up to 5 or 6 percent of the nominal diameter, the length of the periphery of the original circle and the ellipse may be considered to be the same.

On the basis of these assumptions it is possible to calculate the horizontal movement of any point on the periphery of a circular pipe as it deforms to an elliptical shape. The curve obtained by plotting the horizontal movement of each point on the vertical diameter of the pipe is a witch-shaped curve as shown in figure 8.

However, since the horizontal movements of points within the top and bottom 40° segments are small, a simpler parabolic curve embracing the middle 100° of the semicircle may be substituted for the witch without serious error. Since pressures are assumed to be proportional to movement, the passive resistance pressures on each side of the pipe are distributed parabolically over the middle 100° of the pipe; and the maximum unit pressure, which will occur at the ends of the horizontal diameter, is equal to the modulus of passive resistance of the fill material times one-half the horizontal deflection of the pipe.

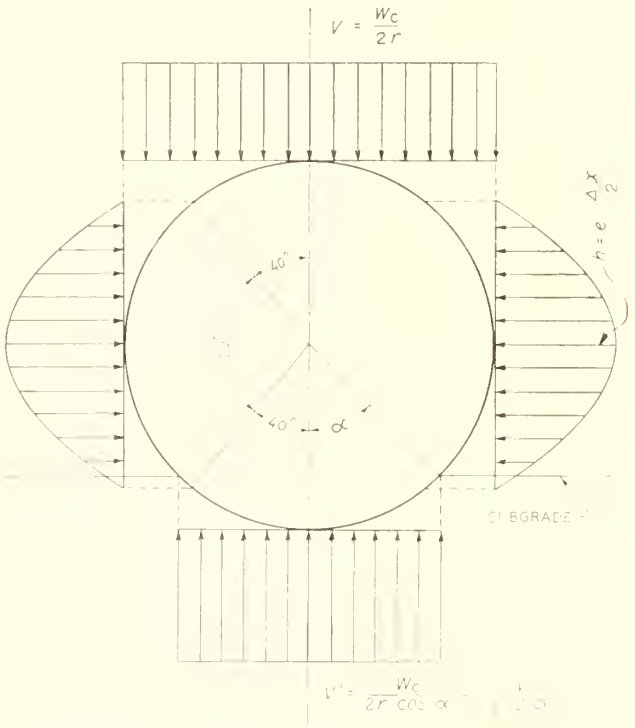

FIGURE 9.—HYPOTHETICAL DISTRIBUTION OF PRESSURE ON A FLEXIBLE CULVERT PIPE UNDER AN EARTH FILL.

This hypothesis may be summarized as follows:

1. The vertical load may be determined by Marston's theory of loads on conduits and is distributed approximately uniformly over the width of the pipe.

2. The vertical reaction is equal to the vertical load and is distributed approximately uniformly over the width of bedding of the pipe.

3. The horizontal pressure on each side of the pipe is distributed parabolically over the middle 100° of the pipe and the maximum unit pressure is equal to the modulus of passive resistance of the fill material multiplied by one-half the horizontal deflection of the pipe.

The distribution of pressures around a flexible pipe under an earth fill in accordance with this hypothesis is shown graphically in figure 9.

Having set up this hypothesis it is possible to develop mathematical expressions for the moments, thrusts, shears, and deflections of a pipe, in terms of the properties of the pipe and the earth of which the embankment is constructed. Referring to figure 10 and calling clockwise moments negative, the equation for the moment at any point is

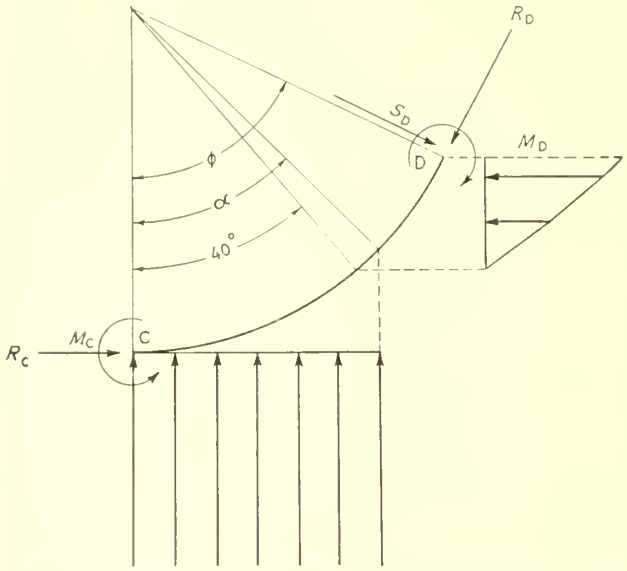

FIGURE 10.—FREE BODY DIAGRAM FOR DETERMINING MOMENT, THRUST, AND SHEAR AT ANY POINT D ON A FLEXIBLE CULVERT PIPE.

$$M=M_C+R_Cr(1-\cos\phi)\Big|_0^{\pi}-0.5v'r^2\sin^2\phi\Big|_0^{\alpha}$$
$$-\sin\alpha r^2v'\left(\sin\phi-\frac{\sin\alpha}{2}\right)\Big|_{\alpha}^{\pi}$$
$$-hr^2(0.147-0.51\cos\phi+0.5\cos^2\phi-0.143\cos^4\phi)\Big|_{40°}^{140°}$$
$$+1.021hr^2\cos\phi\Big|_{140°}^{\pi}-0.500vr^2(1-\sin\phi)^2\Big|_{\frac{\pi}{2}}^{\pi}\text{------(14)}$$

in which

M = moment at any point.
M_C = moment at bottom of pipe.
R_C = tangential thrust at bottom of pipe.
r = mean radius of pipe.
ϕ = the central angle.
v = unit pressure on top of pipe.
v' = unit pressure on bottom of pipe.
h = maximum unit pressure on side of pipe.
α = one-half the bedding angle.

Substituting equation (14) in equation (6) and integrating yields

$$M_C=-R_Cr+0.057vr^2+0.345hr^2$$
$$+v'r^2[0.08\alpha-0.04\sin 2\alpha-0.159\sin^2\alpha(\pi-\alpha)$$
$$+0.318\sin\alpha(1+\cos\alpha)]\text{----------(15)}$$

FORMULA FOR USE IN DESIGN OF FLEXIBLE CULVERT PIPES DEVELOPED

This is the expression for the bending moment at the bottom of the pipe, but contains R_C, the thrust at that point, which is as yet unknown. To evaluate R_C, the displacement theory of arches may be utilized as was done by Cain.[4] In this theory the bottom of the pipe is assumed to be fixed. Then the displacement of any point on the ring relative to the bottom will equal $\frac{1}{EI}$ times the integral of moments at all intermediate points multiplied by the ordinates to the points that are perpendicular to the supposed displacement. Thus the

[4] Stresses and Deflections of Pipe Culverts, by William Cain. Public Roads, vol. 10, no. 9, November 1929.

horizontal movement of A relative to C will equal

$$\frac{r^2}{EI}\int_0^{\pi} M(1-\cos\phi)d\phi \quad \text{(16)}$$

Since the loads are symmetrical about the vertical axis, this expression may be equated to zero, and since

$$\int_0^{\pi} Md\phi=0, \text{ reduces to}$$

$$\int_0^{\pi} M\cos\phi d\phi=0 \quad \text{(17)}$$

By substituting equation (14) in this expression and integrating,

$$R_C=-0.106v'r\sin^3\alpha+0.511hr+0.106vr \quad \text{(18)}$$

Then from equation (15)

$$M_C=-0.049vr^2-0.166hr^2+v'r^2[0.106\sin^3\alpha+0.080\alpha-0.040\sin 2\alpha-0.159\sin^2\alpha(\pi-\alpha)+0.318\sin\alpha(1+\cos\alpha)] \quad \text{(19)}$$

Since $v=\frac{W_c}{2r}$ and $v'=\frac{W_c}{2r\sin\alpha}$, where W_c is the total vertical load on the pipe per unit of length,

$$M_C=-0.166\,hr^2+W_cr\left[0.053\sin^2\alpha+0.040\frac{\alpha}{\sin\alpha}-0.020\frac{\sin 2\alpha}{\sin\alpha}-0.080\sin\alpha(\pi-\alpha)+0.159\cos\alpha+0.135\right] \quad \text{(20)}$$

$$R_C=0.053W_c(1-\sin^2\alpha)+0.511\,hr \quad \text{(21)}$$

Again using the displacement theory of arches, Cain has obtained an equation for the horizontal deflection of a pipe:

$$\Delta X=\frac{2r^2}{EI}\int_0^{\frac{\pi}{2}} M\cos\phi d\phi \quad \text{(22)}$$

Substituting the general value of M as given in equation (14) and integrating

$$\Delta X=\frac{W_cr^3}{EI}\left[0.500\sin\alpha-0.082\sin^2\alpha+0.080\frac{\alpha}{\sin\alpha}-0.160\sin\alpha(\pi-\alpha)-0.040\frac{\sin 2\alpha}{\sin\alpha}+0.318\cos\alpha-0.208\right]-\frac{0.122hr^4}{EI} \quad \text{(23)}$$

Now according to the hypothesis of this paper, the maximum horizontal unit pressure on the sides of the pipe is equal to the modulus of passive resistance of the earth at the sides of the pipe multiplied by one half the horizontal deflection. That is

$$h=\frac{e\Delta x}{2} \quad \text{(24)}$$

Substituting this expression for h in equation (23)

$$\Delta X=\frac{KW_cr^3}{EI+0.061er^4} \quad \text{(25)}$$

in which

ΔX=horizontal deflection of flexible culvert pipe.

$K=0.500\sin\alpha-0.082\sin^2\alpha+0.080\frac{\alpha}{\sin\alpha}-0.160\sin\alpha(\pi-\alpha)-0.040\frac{\sin 2\alpha}{\sin\alpha}+0.318\cos\alpha-0.208$.

W_c=vertical load per unit length of pipe.

r=mean radius of pipe.

E=modulus of elasticity of pipe metal.

I=moment of inertia per unit length of cross section of pipe wall.

e=modulus of passive resistance of the enveloping earth.

This equation is offered tentatively for use in the design of flexible culvert pipes when the conditions of installation are sufficiently well known to justify the calculation of the vertical load on the pipe by means of Marston's conduit load theory, and the modulus of passive resistance of the filling material is known or can be estimated within a reasonable tolerance. Although the equation gives the horizontal deflection of the pipe, both theory and field observations indicate that the vertical and horizontal deflections are nearly the same under most conditions.

Equation (25) is general for all widths of bedding if the reaction is uniformly distributed over the full width of bedding as assumed. Values of the bedding constant K for widths from $\alpha=0$ to $\alpha=90°$ are as follows:

α Degrees	Bedding constant K
0	0.110
15	.108
22½	.105
30	.102
45	.096
60	.090
90	.083

The value of $K=0.083$ for $\alpha=90°$ agrees with the value obtained by Talbott for a pipe loaded uniformly over the top and bottom.[5]

FOUR CULVERTS INSTALLED AND TESTED

In order to test the applicability of this design formula, an experimental program was conducted in which four corrugated metal-pipe culverts were installed at the Iowa Engineering Experiment Station field laboratory at Ames, Iowa, and loaded with a clay embankment 15 feet high above the top of the culverts. The pipes for these experiments and the laboratory studies previously referred to were of four different diameters and United States gage thicknesses, namely: 36-inch, 16-gage; 42-inch, 14-gage; 48-inch, 14-gage; and 60-inch, 12-gage. Lightweight pipes were deliberately chosen so that the diameter changes under the fill loads would be relatively high. All of the pipes tested were furnished without charge by several manufacturers of corrugated pipe.

The embankment was 17 feet wide on top with side slopes about 1.2 to 1. The center 17 feet of the culverts under the full-height portion of the embankment consisted of eight sections each 25½ inches long, and constituted the test sections on which observations were made. The 20 feet of culvert at each end was merely filler pipe under the side slopes of the embankment. The culverts were spaced about 20 feet center-to-center. Figure 11 shows the culverts in place before the fill was constructed.

[5] Tests of Cast-Iron and Reinforced Concrete Culvert Pipe. Bulletin 22, Engineering Experiment Station, University of Illinois, p. 18. Reprinted 1922.

FIGURE 11.—CULVERTS IN PLACE BEFORE THE FILL WAS CONSTRUCTED.

The site chosen for the experimental embankment was the bottom of an old gravel pit which had been partially filled in with strippings of the pit. A subgrade was first prepared by hauling in clay from a borrow pit and making a fill which varied in thickness from about 6 inches on one side to about 1 foot 9 inches on the other. This subgrade was built level for a width of 17 feet in the center and sloped downward 6 inches in 20 feet at each side. The culverts were thus placed higher in the center than at the ends in order to make sure that no surface water would get to the center test sections. The center line of the subgrade ran east and west, the longitudinal axis of each culvert thus being in a north and south line.

After completion of this subgrade, the bedding for the culverts was prepared by cutting a trench having a circular-shaped cross section for the full length of each culvert. The radius of this trench was made 2 inches greater than the pipe to be laid in it and then refilled with sand, which was struck off with a template of the same radius as the pipe. The depth and width of each trench was such that when the pipes were laid they were in contact with the sand bedding for the bottom 90° of the circumference. Thus the pipes projected above the subgrade a distance equal to 0.85 of their diameter, making the projection ratio of Marston's theory, $p=0.85$.

The clay filling material was hauled in two-wheeled scrapers drawn by teams. The fill on each side of the south half of each culvert was hand tamped in 6-inch layers for a distance out from the sides equal to the diameter of the pipe, and for a depth equal to three-fourths the distance the pipe projected above the subgrade. The fill at the sides of the north half of each culvert and at all other places outside this tamped area was simply dumped from the scrapers and placed by shovels without any special manipulation affecting the density or other properties of the embankment. It was expected that the pipes with the untamped side fills would deflect more than those with the tamped side fills and this proved to be the case, as will be shown later. Figure 12 shows the layout of the culvert installation and testing apparatus.

The measurements and observations made during the experiments were directed toward three principal objectives.

1. The settlements of various elements of the pipes and adjacent embankment material was observed in order that the settlement ratio in Marston's theory could be calculated. Also, the unit weight of the fill material was measured and all other necessary data obtained so that the load on the culverts could be calculated according to this theory.

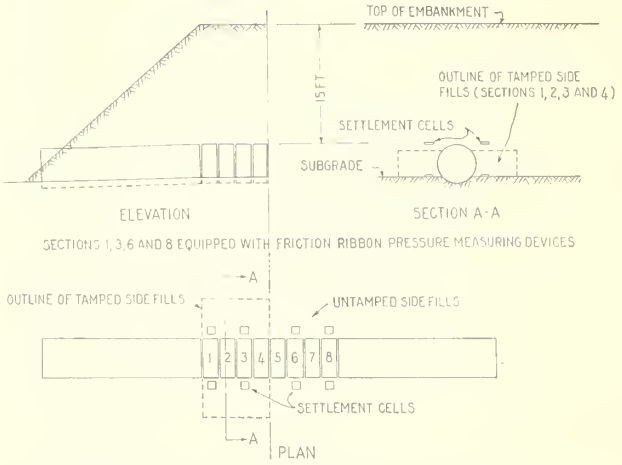

FIGURE 12.—LAYOUT OF CULVERT INSTALLATION AND TESTING APPARATUS.

2. The distribution of vertical load, vertical reaction, and horizontal pressures were measured in order to check the hypothetical distributions and to determine the value of the modulus of passive resistance of the clay filling material, both in the tamped and untamped condition.

3. The vertical and horizontal deflections of the pipes were measured for comparison with the hypothetical deflections computed by equation (25).

Marston's theory of loads on conduits [6] involves certain physical characteristics of both the conduit and the embankment and also a settlement ratio

$$r_{sd}=\frac{(s_m+s_g)-(s_f+d_c)}{s_m} \quad (26)$$

in which

r_{sd} = the settlement ratio.
s_m = the settlement of the columns of earth at the sides of the culvert which extend from the subgrade to the level of the top of the pipe.
s_g = the settlement of the subgrade.
(s_m+s_g) = the settlement of the horizontal plane of the embankment which was level with the top of the pipe at zero fill.
s_f = the settlement of the pipe invert.
d_c = the vertical deflection of the pipe.
(s_f+d_c) = the settlement of the top of the pipe, due to its flexibility plus yielding of its foundation.

The settlements s_m and s_g were measured by means of Ames settlement cells, which are fully described in Bulletin 112 of the Iowa Engineering Experiment Station. There were 64 cells used in all. Half of these were placed on the subgrade at points 1 foot out from the sides of sections 1, 3, 6, and 8 of each culvert and these gave a measure of s_g. The other half were placed at the level of the top of the pipe and directly over the first group. These gave a measure of (s_m+s_g) and by subtracting the value of s_g, the value of s_m was obtained.

DISTRIBUTION OF PRESSURES ON PIPES MEASURED BY FRICTION RIBBONS

The settlements of the pipe inverts, s_f, were measured by means of an ordinary level and rod. By taking short sights, it was possible to read the rod to 0.001 foot with confidence. The shortening of the vertical

[6] The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments, by Anson Marston. Bulletin 96, Iowa Engineering Experiment Station.

diameter, d_c, was measured by means of an inside caliper equipped with a sliding pointer and a steel scale graduated to 0.01 inch. The horizontal deflections were measured with the same instruments. All the deflections were measured between gage points drilled in a valley of the corrugations at the center of each individual pipe section.

The settlement ratios for the various culverts were readily determined from these data and formula (26), and are directly applicable for determining the loads on the pipes with untamped side fills. It is necessary, however, to apply a conversion factor to the settlement ratios for the sections with tamped side fills, since Marston's theory assumes that the modulus of compression of the fill material is uniform throughout the height of the embankment from the subgrade to the top. Where the side fills were tamped, this assumption does not apply, since the compression per unit of load for the tamped material is much less than for untamped material.

It can be shown that the settlement ratios obtained from test data and applicable to the tamped sections must be multiplied by the ratio of the modulus of compression of the tamped fill material to that of the untamped material before they can validly be used to compute the load on the tamped sections by Marston's load formula. Further, the value of s_m is proportional to the modulus of compression of the column of material within the height pB_c, and since this material was tamped adjacent to half of each culvert and untamped at the other half, the ratio of s_m for the tamped and untamped material will be equal to the ratio of the moduli of compression for tamped and untamped material; that is

$$\frac{s'_m}{s_m}=\frac{C'}{C}$$

The values of s_m and s'_m are readily obtainable from the test data. Settlements of the 60-inch culvert with tamped side fills are shown in figure 13, and settlements for the 60-inch culvert with untamped side fills are shown in figure 14.

The distribution of the vertical and horizontal pressures on the pipes was measured by means of stainless steel friction ribbons that were made to slide between two layers of canvas and that passed radially inward over a stainless steel roller at each end of the 25½-inch pipe sections. After the fill was placed, these ribbons were pulled and the pressure normal to the ribbon was assumed to be proportional to the pull required to start the ribbon in motion. The ribbons were ½ inch wide and 0.008 inch thick and very uniform in surface texture. Twenty ribbons were placed on sections 1, 3, 6, and 8 of each culvert, 10 being placed horizontally to measure vertical pressures and the other 10 placed vertically to measure horizontal pressures.

Each ribbon was mounted on either a strip of steel 1 inch wide or on a 1-inch angle that was welded to the pipe sections on top of the corrugations. The steel strips were placed at the top, bottom, and sides of the pipe, while short-legged angles were used at the 22½-degree points and equal-legged angles at the 45-degree points. The inner canvas surface was cemented directly to the steel strip or angle and the outer canvas was sewed around these shapes after the ribbon was placed. After all of the ribbons and canvases were in place, the section was installed in the culvert and surrounded with a sheet of pure gum rubber about one-sixteenth inch thick. These sheets were of sufficient length to go completely around the pipe with a generous overlap, and the ends were securely cemented. Each sheet was 3 feet wide, so the edges overlapped enough to permit

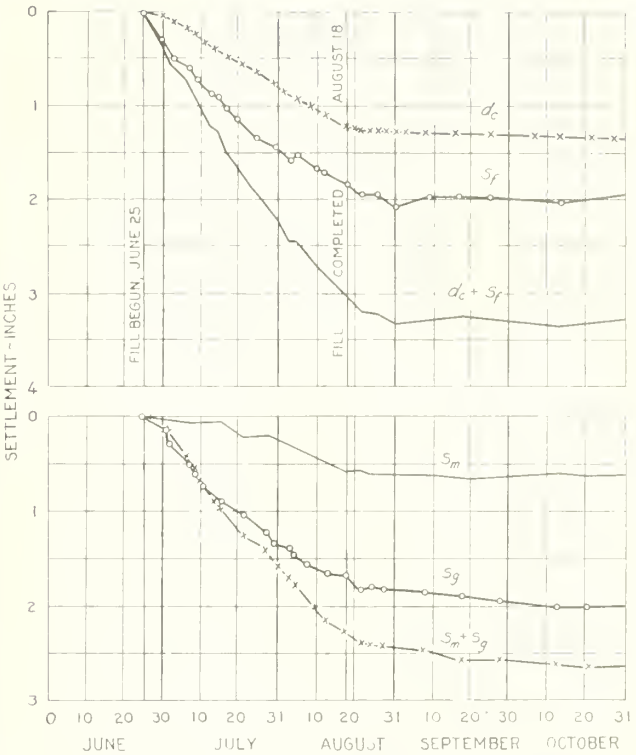

FIGURE 13.—SETTLEMENT OF THE 60-INCH CULVERT WITH TAMPED SIDE FILLS.

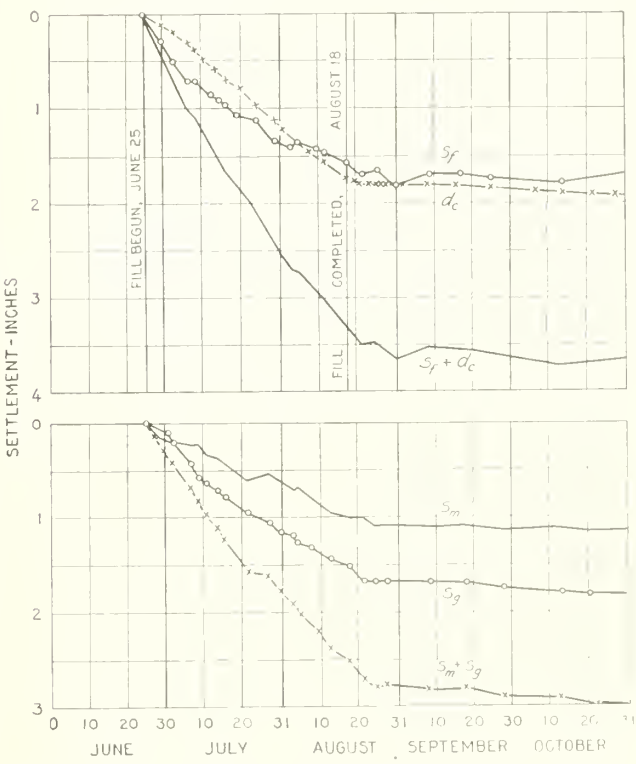

FIGURE 14.—SETTLEMENT OF THE 60-INCH CULVERT WITH UNTAMPED SIDE FILLS.

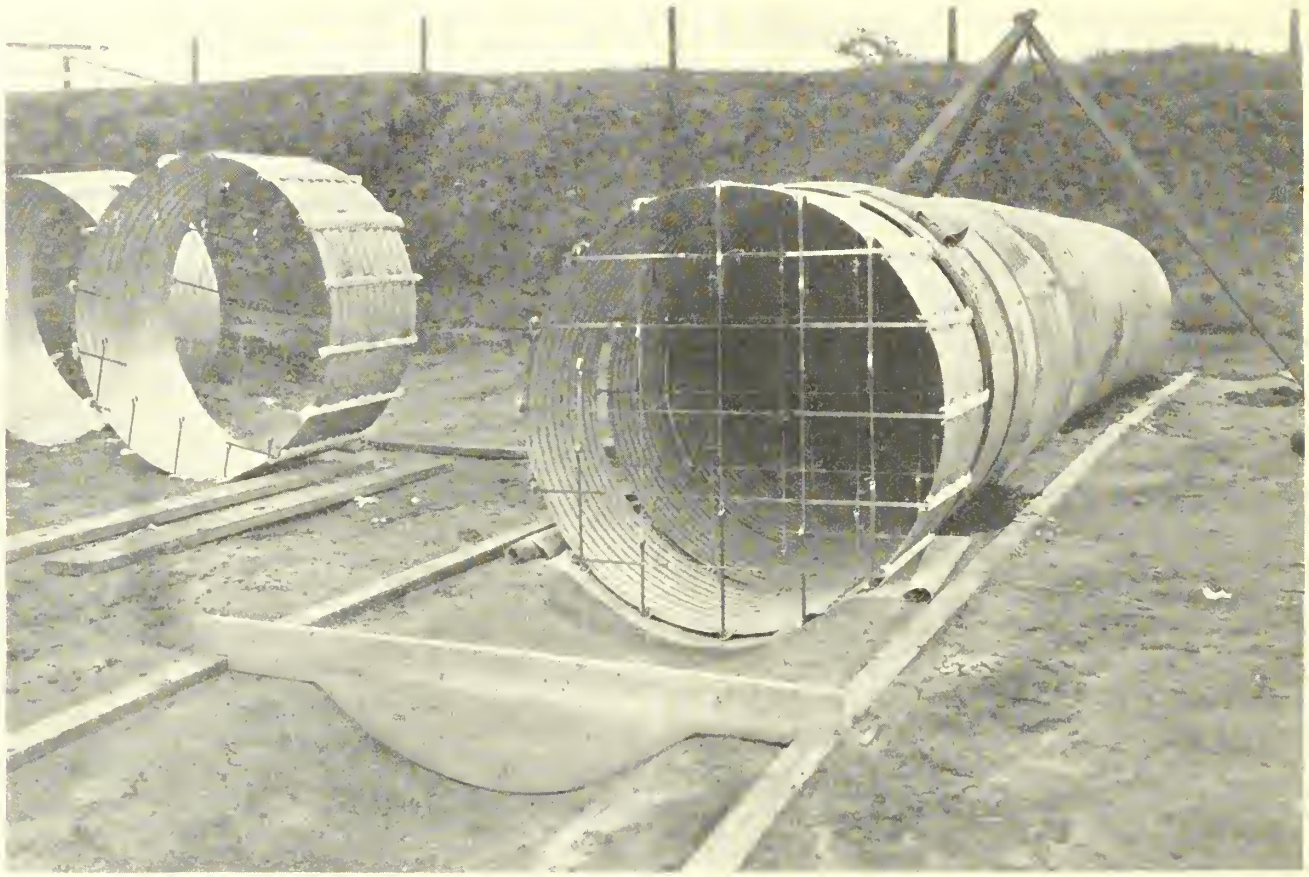

FIGURE 15.—SECTIONS OF CULVERT PIPE ON WHICH RIBBONS ARE BEING INSTALLED TO MEASURE VERTICAL AND HORIZONTAL EARTH PRESSURES.

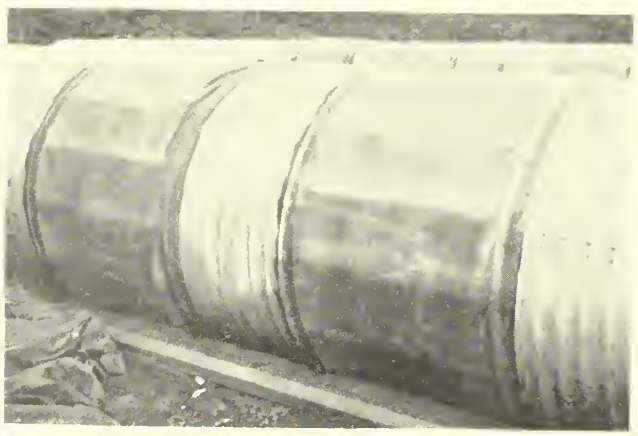

FIGURE 16.—SECTIONS OF CULVERT PIPE ON WHICH RIBBONS HAVE BEEN INSTALLED AND COVERED WITH A PROTECTIVE RUBBER JACKET.

them to be cemented to adjacent culvert sections. The purpose of the rubber jackets was to keep the dirt and moisture of the fill from entering the ribbon spaces. Ribbon-equipped sections during and after installation are shown in figures 15 and 16.

The ribbons have been of great value in these studies and have yielded much information in regard to the distribution of the various pressures, particularly the horizontal pressures, about which so little is known. They are not precision instruments, however, and the results obtained from many tests must be studied before specific statistical conclusions are justifiable.

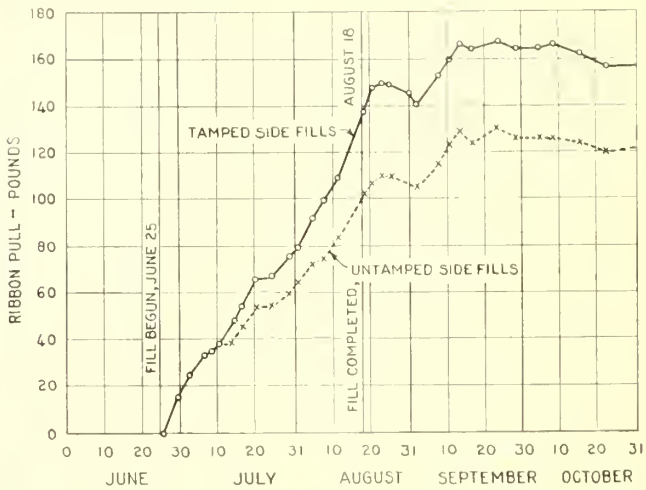

FIGURE 17.—AVERAGE PULL REQUIRED TO MOVE TOP RIBBONS ON 48-INCH PIPE.

Figure 17 shows the average pull required to move the top ribbons at various times during and after fill construction for the 48-inch pipe.

An attempt was made to calibrate the relations between pressure and pull on the ribbons by applying pressure to a ribbon in a similar assembly in the laboratory, but without success. For a given set of conditions, the ratio between pull and pressure remains fairly constant; but when other conditions are imposed, such as different moisture content of the canvas surfaces, the results are widely different. It has been impossible to

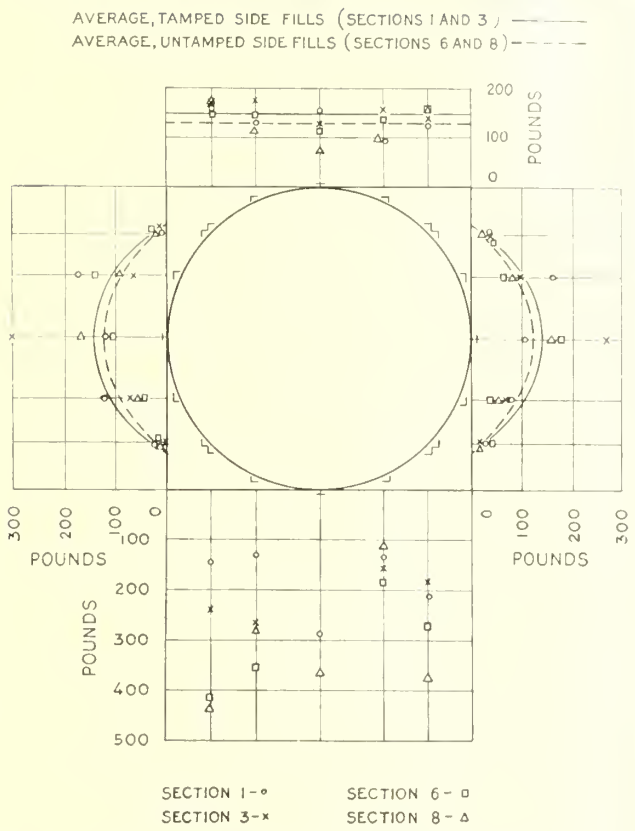

FIGURE 18.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 36-INCH PIPE ON AUGUST 26, 1936.

identify the conditions existing in the field trials with sufficient accuracy to attempt to reproduce them in the laboratory calibration. Instead, the ribbon assemblies on the field experimental culverts have been calibrated by calculating the vertical load on the pipes by means of Marston's load theory, using actual settlement and other data obtained in the experiments. Then, assuming the vertical loads were uniformly distributed over the top 180° of the pipe, the relation between pull on the top ribbons and vertical unit pressure has been obtained.

This assumption of uniform distribution of the vertical load on the pipes is based upon previous studies of loads on rigid pipe and rectangular box culverts, and upon a study of the pulls of top ribbons in these experiments. These pulls, while they varied widely in individual values, showed a definitely uniform distribution trend as indicated by the data shown in figures 18 to 21, inclusive. These data show the pressure distribution patterns on 1 day only, but they are typical of ribbon pulls prior to that date.

MEASUREMENTS MADE FREQUENTLY DURING FILL CONSTRUCTION AND AFTER COMPLETION

The conditions affecting the coefficient of friction between the ribbons and canvas surfaces on the top and the two sides of the pipe were in all probability identical and the calibration of the top ribbons has been applied to the side ribbons. The bottom ribbons, however, presented a different problem. In fact little credence is placed in the results obtained from the bottom ribbons, which were erratic in performance and of questionable reliability. This resulted from the fact that in placing the pipes with the ribbons assembled, it was

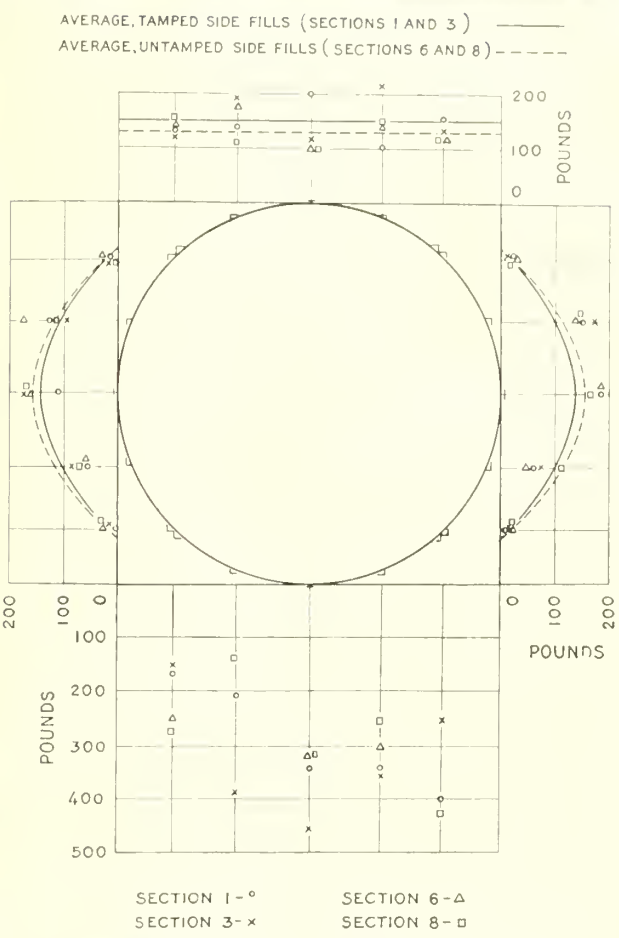

FIGURE 19.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 42-INCH PIPE ON AUGUST 26, 1936.

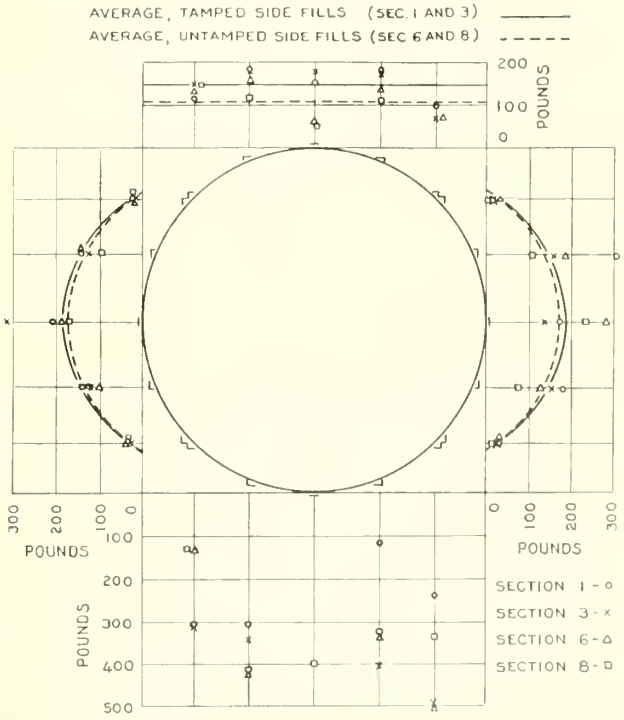

FIGURE 20.— PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 48-INCH PIPE ON AUGUST 26, 1936.

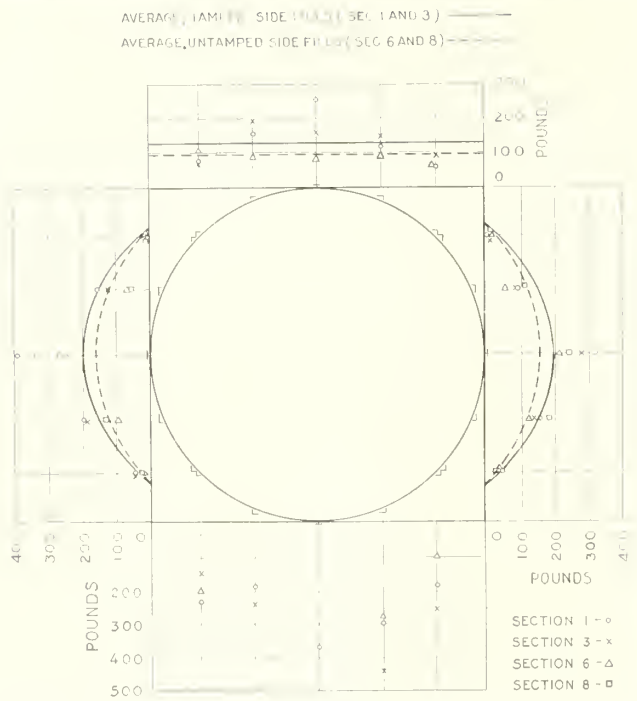

FIGURE 21.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 60-INCH PIPE ON AUGUST 26, 1936.

virtually impossible to prevent small quantities of the bedding sand from entering the ribbon and roller spaces; and although precautions were taken to prevent this, some particles did get in between the ribbons and canvases. Also, during construction of the embankment, when diameter changes and ribbon pulls were being measured every day, some dust and dirt was carried into the culverts on the shoes of the operators and found its way to the roller spaces. The distribution of the vertical reaction on the pipes must, therefore, be judged empirically rather than wholly on the basis of the bottom-ribbon pulls.

The pipes were placed in their beddings and the fill at the sides between the subgrade and the top of the pipes was placed during the first 3 weeks in June 1936. The fill over the top of the pipes was begun on June 25 and was completed to a height of 15 feet on August 18, 1936. During this loading period the settlements, deflections, and ribbon pulls were observed at least once for every 1-foot increment of fill. The observations were continued after completion of the fill at about weekly intervals until about October 1, and since then at less frequent intervals.

Some of the data that have been selected as typical for the period from June 25 to October 31 are shown graphically in figures 13, 14, and 17 to 23. The settlements of various elements of the embankment and the pipes of the 60-inch culvert with tamped and untamped side fills are shown in figures 13 and 14. The points through which the curves are drawn are the average of 4 settlement-cell readings for (s_m+s_g) and s_g, and vertical diameter change and invert settlement of 4 pipe sections for d_c and s_f. The average pull required to move the top ribbons on the 48-inch culvert is shown in figure 17 both for the tamped and untamped side fills. Here each point is the average of 10 ribbon pulls, 5 on each of 2 sections.

It appears from examination of the deflection and foundation-settlement curves that the maximum load

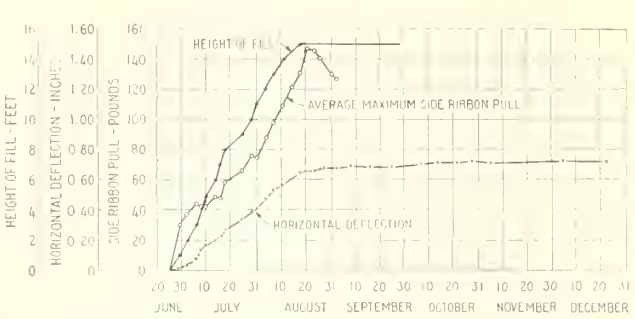

FIGURE 22.—HORIZONTAL DEFLECTIONS AND SIDE-RIBBON PULLS FOR THE 36-INCH PIPE WITH TAMPED SIDE FILLS.

on the culverts was reached within a very short time after the fill was completed, probably within less than a week. The ribbon pull, for the sections in untamped fills, however, shows a slight increase early in September. During the summer of 1936 there was a drought in the corn belt and the rainfall at Ames was practically negligible from the middle of June to September 4, accompanied by extremely hot and dry atmospheric conditions. On September 4 more than 4 inches of rain fell and was followed by more nearly normal atmospheric conditions. In the attempts to calibrate the ribbon assemblies in the laboratory, it was found that moisture in the canvas increased the starting pull on the ribbons and in all probability the increased pull on the field ribbons following this heavy rain resulted from increased moisture rather than from an increase in load on the pipes. A portion of the increase in ribbon pull may have been caused by the weight of water absorbed by the fill during this rainfall.

With these conditions in mind, the data taken on August 26 has been chosen as representative of the situation after the maximum load on the culverts was reached and before the ribbon pulls were affected by the change in meteorological conditions following the rain of September 4. The ribbon pulls on the top and side ribbons on August 26 are shown in figures 18 to 21. The patterns of distribution of pressure shown in these figures are typical of the patterns until the rain of September 4. After this date, the distribution of the pulls on the side ribbons became somewhat erratic.

In these ribbon-pull diagrams, lines have been drawn representing the average ribbon pulls for sections having tamped side fills and for sections having untamped fills. In the case of the top ribbon the readings were averaged numerically in accordance with the uniform pressure assumption. For the side ribbons, the readings have been weighted to obtain the average curves in accordance with the assumed parabolic distribution. The values of the maximum ordinates to the weighted average curves for the 36-inch culvert throughout the loading period are shown in figures 22 and 23.

TAMPING SIDE FILLS DOUBLED THEIR CAPACITY TO RESIST CULVERT DEFLECTION

In order to determine the relations between the lateral pressure on the pipes and the horizontal movement of the sides of the pipes, that is, the modulus of passive resistance of the fill material, it is necessary to interpret the pulls required to start the side ribbons in terms of pressure. This has been done in the following manner, since the laboratory calibration operations failed to produce results that could be applied to the field ribbons.

First the load on each experimental culvert was calculated by means of Marston's conduit load theory

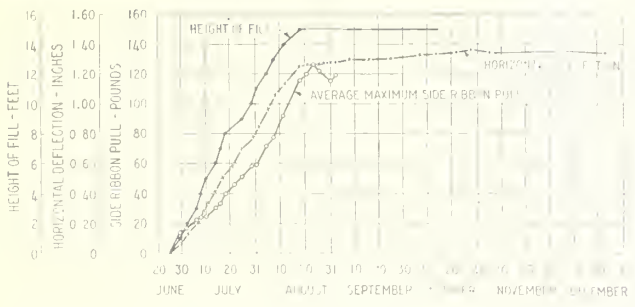

FIGURE 23.—HORIZONTAL DEFLECTIONS AND SIDE-RIBBON PULLS FOR THE 36-INCH PIPE WITH UNTAMPED SIDE FILLS.

using measured settlement data and the measured unit weight of the clay filling material (120 pounds per cubic foot). The load was divided by the area of the horizontal projection of the pipe to obtain the average vertical pressure on the pipe. This unit pressure was then divided by the average ribbon pull on the top ribbons to obtain the pressure on the ribbons per pound of pull required to start the ribbons. The test data and calculated results of these operations for all the culverts are shown in table 2. The average value of this ratio of unit pressure to pull is 0.0673 pound per square inch per pound of pull.

Having this ratio, and assuming that the conditions affecting it were the same for the side ribbons as for the top ribbons, the side-ribbon pulls may be multiplied by it to obtain the magnitude and distribution of the lateral pressure. Further, the modulus of passive resistance of the fill material may be obtained by dividing the maximum lateral unit pressure by ½ the horizontal deflection of the pipe. The data resulting from these calculations are given in table 3, and show an average modulus of passive resistance for the untamped clay in this fill to be 13.4 pounds per square inch per inch of movement, and 27.0 pounds per square inch per inch for the tamped filling material. It appears, therefore, that tamping the side fills in these experiments practically doubled their capacity to assist the pipes to carry the vertical earth load.

Having determined the modulus of passive resistance of the fill material and knowing the physical properties of the pipes, the deflection of the pipes under the calculated load can be determined by means of tentative design formula (25). The results of these calculations are shown in table 4. The measured horizontal and vertical deflections on August 26 are also shown for comparison. The calculated horizontal deflections show fairly close agreement with the actual measured values, thus serving to substantiate the load hypothesis given earlier in this paper.

TABLE 3.—*Moduli of passive resistance for tamped and untamped side fills for the various culverts tested*

Diameter of culvert [1]	Maximum ordinate of weighted average pull curve	Maximum lateral pressure	$\frac{\Delta x}{2}$	Modulus of passive resistance	
				Tamped side fills	Untamped side fills
Inches	*Pounds*	*Lb./sq. in.*	*Inch*	*Lb./sq. in. /in.*	*Lb./sq. in. /in.*
36 T	141	9.49	0.335	28.30	
36 U	122	8.21	.635		12.93
42 T	141	9.49	.385	24.67	
42 U	153	10.30	.695		14.82
48 T	187	12.59	.430	29.30	
48 U	173	11.65	.810		14.39
60 T	193	12.99	.505	25.71	
60 U	154	10.37	.895		11.60
Average				27.00	13.43

[1] T indicates tamped side fills, U indicates untamped side fills.

TABLE 4.—*Calculated horizontal deflections and measured horizontal and vertical deflections of the various pipes*

Diameter of culvert [1]	Calculated horizontal deflection		Measured horizontal deflection [2]		Measured vertical deflection [2]	
	Deflection	Percentage of nominal diameter	Deflection	Percentage of nominal diameter	Deflection	Percentage of nominal diameter
Inches	*Inches*		*Inches*		*Inches*	
36 T	0.83	2.31	0.67	1.86	0.71	1.97
36 U	1.17	3.25	1.27	3.53	1.21	3.36
42 T	.94	2.24	.77	1.83	.84	2.00
42 U	1.41	3.35	1.39	3.31	1.31	3.12
48 T	.97	2.02	.86	1.79	1.02	2.12
48 U	1.56	3.25	1.62	3.38	1.56	3.25
60 T	.87	1.45	1.01	1.68	1.26	2.10
60 U	1.72	2.88	1.79	2.98	1.82	3.03

[1] T indicates tamped side fills, U indicates untamped side fills.
[2] On August 26, 1936.

These calculations reveal the relative influence of the inherent strength of the pipe and the side restraint afforded by the fill upon the deflection the culverts assumed under the fill. In the design formula (25), the term EI in the denominator represents the influence of the pipe strength and the second term $0.061\ cr^4$ represents the influence of the passive resistance of the fill material. In these experiments, the second term was 1.65 times the first term for the 36-inch pipe with untamped side fills and this ratio varied up to a

TABLE 2.—*Test data and ratio of calculated unit pressure to pull on ribbons for the various culvert pipes tested*

Diameter of culvert [1]	(s_m+s_g)	s_g	s_m or s'_m	d_c	s_f	(d_c+s_f)	r_{td}	Ratio $\frac{s'_m}{s_m}$	Adjusted r_{td}	Load on pipe			Average pull on top ribbon	Ratio of unit pressure to pull
										Lb. per lin. ft.	*Lb. per lin. in.*	*Lb. per sq. in.*		
Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*							*Pounds*	*Lb. per sq. in. per lb. pull*
36 T	2.26	1.74	0.52	1.71	1.71	2.42	−0.31	0.57	−0.18	4,100	342	9.08	144	0.0630
36 U	1.91	1.00	.91	1.21	1.05	2.26	−.38			3,600	300	7.84	127	.0617
42 T	2.29	1.61	.68	.84	1.60	2.45	−.24	.56	−.14	5,100	425	9.71	150	.0647
42 U	2.21	1.00	1.21	1.31	1.21	2.52	−.26			4,500	375	8.45	130	.0650
48 T	2.13	1.42	.71	1.02	1.33	2.35	−.31	.56	−.17	5,500	458	9.18	149	.0616
48 U	2.00	.91	1.09	1.56	.86	2.42	−.38			4,900	408	8.06	110	.0732
60 T	2.41	1.81	.60	1.27	1.94	3.21	−1.33	.57	−.76	5,800	483	7.79	127	.0614
60 U	2.77	1.67	1.10	1.82	1.68	3.50	−.67			6,100	508	8.09	92	.0880
Average														.0673

[1] T indicates tamped side fills; U indicates untamped side fills.

maximum of 16.05 for the 60-inch pipe with tamped side fills. From these facts it seems probable that pipes of greater thicknesses would have deflected but slightly less than did the pipes actually used, especially the larger sizes. On the other hand, greater consolidation of the fill material, by compaction at optimum moisture content or by some artificial stabilization process, might have reduced the deflections materially.

FORMULAS GIVEN FOR MOMENT, THRUST, AND SHEAR

Although the deflection of flexible culvert pipe is the phenomenon of primary interest from the structural standpoint, because failure of flexible pipes occurs by excessive deflection rather than excessive stress, the stress distribution around the periphery of the pipes may also be of some interest. From equations (20) and (21) the moment and thrust at the bottom of the culvert pipes in these experiments ($\alpha=45°$) are

$$M_C=0.157W_c r-0.166hr^2 \quad (27)$$
$$R_C=0.026W_c+0.511hr \quad (28)$$

Having the moment and thrust at the bottom of the pipe, the moment, thrust, and shear at any other point on the ring may be obtained from the equations of equilibrium. It is convenient to write the equations for these quantities for vertical loads and for horizontal loads separately.

For moment, thrust, and shear caused by vertical loads—

When ϕ lies between 0 and 45°:

$$M_D=W_c r(0.183-0.026\cos\phi-0.354\sin^2\phi) \quad (29)$$
$$R_D=W_c(0.026\cos\phi+0.707\sin^2\phi) \quad (30)$$
$$S_D=W_c(0.707\sin\phi\cos\phi-0.026\sin\phi) \quad (31)$$

When ϕ lies between 45° and 90°:

$$M_D=W_c r(0.360-0.026\cos\phi-0.500\sin\phi) \quad (32)$$
$$R_D=W_c(0.026\cos\phi+0.500\sin\phi) \quad (33)$$
$$S_D=W_c(0.500\cos\phi-0.026\sin\phi) \quad (34)$$

When ϕ lies between 90° and 180°:

$$M_D=W_c r(0.110-0.026\cos\phi-0.250\sin^2\phi) \quad (35)$$
$$R_D=W_c(0.026\cos\phi+0.500\sin^2\phi) \quad (36)$$
$$S_D=W_c(0.500\sin\phi\cos\phi-0.026\sin\phi) \quad (37)$$

For moment, thrust, and shear caused by horizontal loads—

When ϕ lies between 0 and 40°:

$$M_D=hr^2(0.345-0.511\cos\phi) \quad (38)$$
$$R_D=0.511hr\cos\phi \quad (39)$$
$$S_D=0.511hr\sin\phi \quad (40)$$

When ϕ lies between 40° and 140°:

$$M_D=hr^2(0.199-0.500\cos^2\phi+0.143\cos^4\phi) \quad (41)$$
$$R_D=hr(\cos^2\phi-0.568\cos^4\phi) \quad (42)$$
$$S_D=hr(\sin\phi\cos\phi-0.568\sin\phi\cos^3\phi) \quad (43)$$

When ϕ lies between 140° and 180°:

$$M_D=hr^2(0.345+0.511\cos\phi) \quad (44)$$
$$R_D=-0.511hr\cos\phi \quad (45)$$
$$S_D=0.511hr\sin\phi \quad (46)$$

Moment diagrams for these equations are shown in figure 24 in terms of $W_c r$ for vertical loads and hr^2 or $\frac{e\Delta x r^2}{2}$ for horizontal loads. The net moment is the algebraic sum of the moments caused by the vertical and horizontal loads. Using the calculated loads and deflections and the moduli of passive resistance for tamped and untamped materials, the net flexural stress in the ring of each experimental culvert is as shown in figure 25.

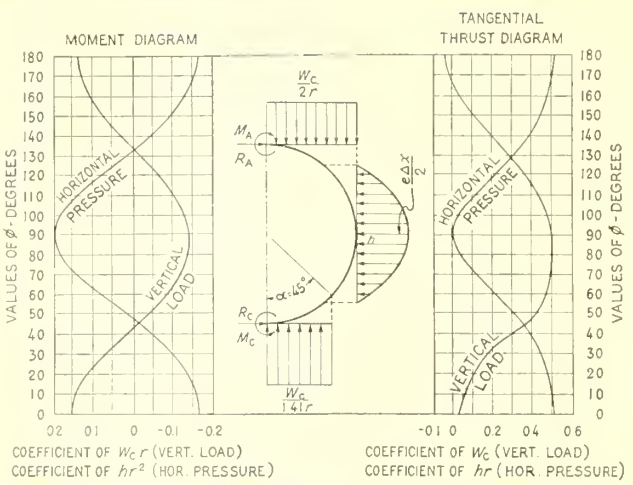

FIGURE 24.—MOMENT AND TANGENTIAL THRUST DIAGRAMS FOR FLEXIBLE CULVERT PIPES.

Since the reaction is less distributed than the vertical load, the greatest bending stress occurs at the bottom of the pipe. This stress diminishes rapidly as ϕ increases, and reverses direction at about 35° to 40° and attains a maximum negative value at about 60°. From 60° to 140° the stresses are relatively low. Beyond this point the stress increases rapidly to the maximum value at the top which is about ⅔ to ¾ the magnitude of the stress at the bottom.

The calculated maximum stresses in the pipes are rather high and in 5 of the 8 culverts they exceed the elastic limit of the metal as determined on companion specimens in the laboratory tests described earlier in this report. However, the length of arc overstressed is relatively short and no appreciable effect on the rate of deflection of the pipe is discernible.

The tangential thrust in the pipe walls is of interest in connection with the design of riveted or bolted longitudinal joints connecting the corrugated sheets of which the pipes are fabricated. The hypothetical loads yield thrust diagrams as shown in figure 24. Using the calculated values of W_c, e, and Δx, the net thrusts per inch length for each experimental pipe have been calculated and are shown in figure 26. The radial shears are small and of little importance in the design of this type of structure.

DEFLECTION CONTINUED AFTER FILL COMPLETION

A phenomenon of considerable interest that has been measured in this and an earlier study of corrugated pipe culverts is that the pipes continue to deflect slowly long after the fill is completed and the maximum vertical load has been attained. In these experiments the pipes have continued to deflect for more than a year after the completion of the fill. The average amount of this additional vertical deflection of the pipes with tamped side fills has been 0.25 percent of the nominal diameter between August 26, 1936 and September 15, 1937, and 0.38 percent for the pipes with untamped side fills.

This same phenomena has been observed in another experiment conducted at Ames in which a 42-inch, 10-gage pipe has been loaded under a 15-foot fill of loam soil for a period of nearly 10 years. This culvert is mounted on a system of weighing platforms and levers so that the vertical load on it can be determined at the time deflections are measured. The load and deflection history of this culvert, which was constructed with untamped side fills, is given in table 5.

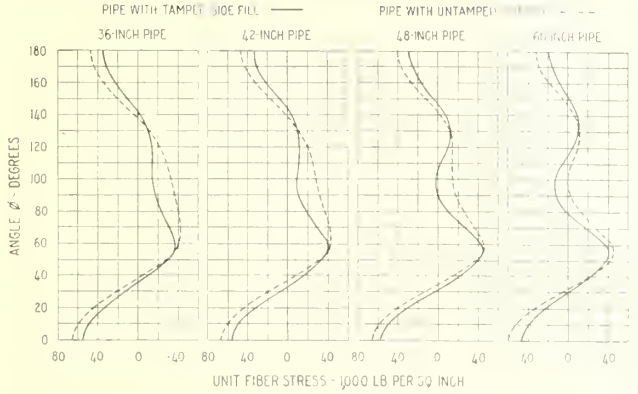

FIGURE 25.—NET FLEXURAL STRESS IN RINGS OF CULVERT PIPE TESTED.

TABLE 5.—*Load and deflection data for a 42-inch culvert pipe under a 15-foot fill of untamped loam soil*

Date	Load	Vertical deflection	
		Deflection	Percentage of nominal diameter
	Lb. per lin. ft.	*Inches*	*Percent*
Dec. 16, 1927 (fill completed)	5,300	1.43	3.41
July 1, 1928	6,000	1.78	4.24
Aug. 15, 1932	6,800	2.44	5.81
Apr. 19, 1935	6,300	2.59	6.17
Sept. 15, 1937	6,400	2.62	6.24

The fact that flexible pipes continue to deform slowly after the fill is completed is in all probability analogous to the widely known fact that all structures resting on earth foundations continue to settle long after the maximum load on the footings is applied. Apparently the fill material at the sides of the pipes slowly recedes in response to the pressure acting between the pipe and the fill and this permits the pipe to deflect even when there is no increase in the vertical load. This phenomenon may be of importance in supplying data upon which specifications for flexible pipe can be based. The deflection at the time of fill completion should possibly be specified for practical reasons; but this specified deflection should be related to the ultimate deflection, which may occur years later and which determines the load capacity of the pipe.

NEED FOR FURTHER STUDIES INDICATED

One of the most important contributions of this research has been the knowledge gained regarding the relationship between the pressure developed at the sides of the pipes and the deflection, but there are many important elements of this relationship that need further study. In these experiments, the straight-line increase of both deflections and side pressures as the fill was constructed justifies the use of a constant ratio between deflection and pressure in this ordinary method of construction.

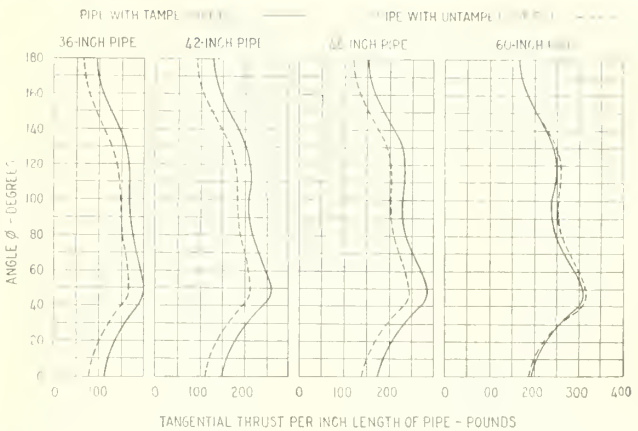

FIGURE 26.—CALCULATED TANGENTIAL THRUSTS IN EACH OF THE PIPES TESTED.

It should be pointed out, however, that each increment of deflection and side pressure was accompanied by an increment of fill or vertical pressure on both the culvert and the material at the sides of the pipe, and the effect of the vertical pressure on the ratio of deflection to side pressure is not revealed in these studies. Therefore, when a flexible pipe is "strutted" before the fill is placed, as is frequently done when high fills are encountered, and the struts are removed after completion of the fill, a different situation is presented and in all probability the ratio between deflection and side pressure will not be a constant in this case.

Further, there is much to be learned in regard to the characteristics of the side fill materials and their effect on the modulus of passive resistance. In these experiments, it was shown that tamping the side fills approximately doubled the modulus, with a consequent reduction in deflection of the pipe. Probably if the tamping had been done under optimum moisture conditions, a greater density and a greater modulus might have been attained.

The width of bedding of all the pipes in these studies was the same, the value of the angle α being 45 degrees. Further experiments should be conducted to obtain data on the effect of other bedding widths.

The hypothesis advanced in this paper has been sufficiently sustained by the experiment that a program to correlate the hypothesis with actual field performance would seem to be justified. In any such program, the observation of field culverts should begin at the time the subgrade for the pipe is prepared so that bedding conditions could be noted in detail and settlement plates installed on the subgrade and at the level of the top of the pipe for determination of practical design values of the settlement ratio. Then deflections of the pipe should be measured at suitable intervals during construction of the fill and after its completion. Also, the characteristics of the side fills should be observed in detail in an attempt to develop working values of the modulus of passive resistance of various types of fill materials.

EIGHTH INTERNATIONAL ROAD CONGRESS TO BE HELD IN JUNE

The Eighth International Road Congress will be held in June 1938 at The Hague, Netherlands, under the auspices of the Permanent International Association of Road Congresses. The Netherlands Government has invited all members of the Association to attend.

The sessions of the Congress are tentatively scheduled to begin on Monday, June 20, and to extend through Saturday, June 25. All-day excursions are planned for Monday, June 27, through Friday, July 1. A trip through The Hague and its environs and the ceremonial closing session of the Congress are tentatively scheduled for Saturday, July 2. Definite and more detailed information as to the schedule of the Congress will be announced later by the Association.

An exhibition of road equipment and materials, several receptions, and numerous excursions are planned. For the ladies who do not want to attend the sessions, separate excursions or visits will be organized for June 21, 22, and 23.

Persons desiring to attend the Congress can obtain temporary membership in the Association by paying a fee of 150 French francs ($4.90 at the rate of exchange on Jan. 25, 1938). Applications and fees can be sent to Mr. Thos. H. MacDonald, Chairman, American National Committee of the Permanent International Association of Road Congresses, Pan American Building, NW., Washington, D. C.

Association members are entitled to attend all sessions of the Congress as well as the receptions and official excursions taking place during the Congress. They are privileged to use the travel facilities and tariff reductions granted on behalf of the Congress. They receive, before the Congress opens, the official reports on the subjects dealt with at the sessions, and after the Congress a general account of the work accomplished.

The following program of subjects has been announced:

FIRST SECTION (CONSTRUCTION AND MAINTENANCE)

First Question:

a. Progress, since the Congress at Munich, with the use of cement for carriageway surfacings.

b. Brick surfaces.

c. Surfaces in special materials such as cast-iron, steel, rubber.

Second Question:

Progress since the Congress at Munich in the preparation and use of:

a. Tar.

b. Bitumen (asphalt).

c. Emulsions. For the construction and maintenance of carriageways.

SECOND SECTION (USE, REGULATION, AND ADMINISTRATION)

Third Question:

Accidents on roads:

a. Bases of statistical returns and their unification.

b. Methods of investigation into the causes of accidents and means for their prevention.

Fourth Question:

The segregation of the various classes on the highway:

Carriageways (single and dual).

Cycle tracks.

Footways.

Service roads in connection with "Ribbon Development," parking places.

Road junctions and crossings:

a. A study of the circumstances which make these provisions desirable or undesirable.

b. Application to motor roads.

FIRST AND SECOND SECTIONS (COMBINED)

Fifth Question:

Examination and standardization of carriageway surfacings from the point of their:

a. Slipperiness or rugosity and resistance to skidding.

b. Light value or the degree to which they absorb light (under artificial illumination).

Sixth Question:

Examination of the subsoil of roads:

a. Determination of the properties of subsoils: Methods of testing, and testing apparatus.

b. Influence of the properties of the subsoil on the construction of roads (foundations and surfaces) and their maintenance.

The Association makes no provision for sending the reports and proceedings of its congresses to persons other than members. All persons, libraries, and organizations, desiring to receive these reports and proceedings should therefore apply for membership.

Reductions in fares for travel in the Netherlands have been offered to Congress members by certain railways, shipping companies, and air lines in the Netherlands. The Association is attempting to obtain for Congress members reductions in fares from railroads and shipping companies of various other countries.

Everyone attending the Congress will be able to find suitable hotel accommodations at The Hague. Several hotels will grant special reductions to Congress members. Hotel accommodations will be available at reasonable rates; for room and breakfast, $1.40 to $3.40 per day; for room and full board, $2.25 to $5.60 per day; room with bath will cost an additional 60 to 90 cents per day. Modest but good accommodations can be obtained at lower prices at numerous boarding houses.

The planned excursions will be 1-day trips or less, beginning and ending at The Hague, so the excursion-

ists need not carry luggage. Whole-day excursions can be expected to cost from $2.80 to $3.40, while short excursions will cost from $0.90 to $1.40.

Many places of particular interest to Congress members will be visited, and ample time will be afforded for careful inspection and study of the more important features. Excursion itineraries will include visits to roads built in lowlands on very poor subgrades, inspection of many large bridges and of tunnel construction at Rotterdam, and observations of the work of reclaiming the famed Zuiderzee. An excursion will be made to the big dike that cuts off the Zuiderzee from the outer sea and forms a new motor road between the northeastern provinces and the heart of the country.

Numerous main highways and secondary roads will be inspected, so that Congress members can get a general idea of the progress made in solving road problems in the Netherlands. If possible a trip will be made at night over the highway from Amsterdam to Haarlem, which is lighted the entire distance.

General sightseeing trips will also be made, including visits to the cheese market at Alkmaar, flower and tree nurseries, quaint towns along the Zuiderzee coast, the Dutch moorland, etc.

Exhibits will show what has been done in the Netherlands during the past few years to improve the network of roads as well as to regulate and safeguard traffic.

The Seventh International Road Congress, held in Munich, Germany, in September 1934, was attended by approximately 2,000 delegates from 50 countries.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,122,820	$ 551,410	64.3	$ 2,710,041	$ 1,355,020	124.6	$ 3,099,020	$ 1,549,055	141.6	$ 4,391,695
Arizona	1,673,415	1,212,650	81.4	1,065,082	704,210	49.5	888,078	597,388	54.1	605,575
Arkansas	2,879,864	2,876,323	183.8	1,281,294	1,277,611	85.9	23,505	23,177		2,221,807
California	6,649,769	3,676,359	149.9	7,085,194	3,720,044	116.0	1,832,467	982,813	32.2	530,038
Colorado	3,172,067	1,771,163	119.8	629,200	341,560	27.6	1,056,150	572,398	37.2	1,710,005
Connecticut	783,198	389,196	9.3	231,520	108,123	.7				1,513,589
Delaware	463,922	231,872	18.1	70,588	35,294	2.8	579,739	265,811	17.7	960,035
Florida	343,682	171,253	14.7	2,763,372	1,381,686	60.2	772,270	386,135	12.2	2,478,109
Georgia	2,113,634	1,024,810	122.5	4,485,023	2,242,507	203.4	2,879,354	1,439,677	126.4	3,921,543
Idaho	2,593,499	1,532,132	198.9	743,890	443,836	56.1	744,295	443,043	43.3	553,955
Illinois	9,986,641	4,930,085	284.4	5,375,161	2,683,473	152.5	6,001,300	3,000,650	86.7	363,321
Indiana	5,683,691	2,823,161	131.9	2,696,415	1,348,207	86.2	1,872,300	936,150	44.3	1,543,176
Iowa	6,554,603	2,968,790	210.2	4,302,663	1,924,718	127.8	2,412,244	1,122,150	68.5	71,630
Kansas	3,838,259	1,916,953	203.9	2,930,392	1,465,097	79.3	1,874,305	937,154	108.1	3,013,055
Kentucky	2,096,616	998,308	64.0	3,611,784	1,805,892	97.3	1,414,098	707,049	80.0	2,173,749
Louisiana	381,379	185,898	9.3	6,555,335	1,430,435	45.2	5,547,243	892,045	23.0	1,688,814
Maine	1,869,761	934,879	52.6	1,908,936	954,468	42.2	549,530	274,760	13.5	118,081
Maryland	930,330	465,145	13.4	1,814,594	905,534	28.9	460,360	230,180	.5	1,493,949
Massachusetts	4,403,913	2,201,955	20.2	1,828,245	914,122	3.3	705,420	352,710	5.0	1,619,545
Michigan	6,459,043	3,167,692	166.9	5,421,010	2,711,005	130.8	1,734,971	792,660	25.3	50,529
Minnesota	6,209,382	3,090,385	289.6	2,783,053	1,380,282	120.1	2,061,662	1,020,831	103.9	1,029,238
Mississippi	2,199,190	1,096,851	102.0	4,319,990	2,204,590	200.0	1,487,740	738,620	58.4	2,593,115
Missouri	8,074,930	4,014,743	443.3	4,462,675	2,127,557	120.5	3,541,856	1,379,471	74.7	1,317,097
Montana	3,824,816	2,144,049	277.7	1,846,412	1,038,231	86.4	571,734	321,594	33.8	1,986,038
Nebraska	2,365,131	1,182,565	239.8	4,418,217	2,196,292	409.2	2,984,508	861,941	118.4	1,902,067
Nevada	2,350,747	2,023,616	106.8	698,358	601,695	62.3	82,190	71,270	5.2	673,602
New Hampshire	360,696	177,963	6.5	480,084	237,150	9.5	311,182	154,753	5.0	864,953
New Jersey	1,674,608	760,009	20.2	1,669,700	834,530	12.5	1,367,360	682,795	7.5	1,209,232
New Mexico	2,940,681	1,802,689	229.0	2,084,983	1,371,538	99.8	352,746	187,444	16.8	199,386
New York	14,161,686	6,506,706	238.5	12,141,954	6,024,460	206.8	2,523,490	1,247,102	42.2	263,327
North Carolina	4,768,607	2,379,740	372.3	4,360,915	2,010,673	141.8	2,682,883	1,306,392	132.0	1,435,123
North Dakota	1,035,479	1,031,251	186.5	1,310,220	1,289,710	77.4	955,709	955,700	59.2	2,445,572
Ohio	3,673,133	1,766,053	51.0	9,122,269	4,524,626	97.4	1,651,515	826,193	19.4	4,987,905
Oklahoma	2,918,620	1,523,667	143.6	2,736,560	1,414,738	114.2	1,492,074	753,183	59.7	2,964,543
Oregon	3,616,277	2,133,704	126.9	1,722,209	1,011,655	83.8	662,848	404,440	16.0	738,817
Pennsylvania	11,604,873	5,774,005	164.8	6,686,538	3,323,829	94.1	3,377,053	1,687,489	58.7	2,089,737
Rhode Island	824,252	403,311	7.1	1,045,330	522,665	14.0	8,930	4,465		774,458
South Carolina	3,295,514	1,377,202	246.4	4,856,415	2,055,705	223.1	1,498,948	652,023	60.2	771,773
South Dakota	2,462,981	1,392,121	247.0	1,638,066	905,760	173.5	467,843	257,723	58.3	2,991,657
Tennessee	1,846,076	922,109	69.4	1,367,384	683,692	40.6	1,778,040	889,020	57.2	4,048,696
Texas	11,039,999	5,506,327	770.5	10,897,080	5,418,952	517.2	3,478,283	1,686,449	221.3	3,805,476
Utah	1,329,852	947,399	130.0	775,530	554,172	61.0	187,080	133,304	1.4	977,044
Vermont	1,016,663	489,925	29.2	1,679,730	739,095	43.1	6,730	3,169		61,552
Virginia	2,930,010	1,452,923	140.7	2,473,147	1,197,158	54.4	3,301,692	1,645,796	96.0	689,929
Washington	1,953,223	1,009,420	75.1	3,323,312	1,743,512	42.3	533,365	280,300	8.7	460,932
West Virginia	1,226,281	616,845	39.3	1,336,886	711,081	30.2	555,241	434,280	15.1	1,930,947
Wisconsin	8,040,395	3,854,287	269.4	4,701,047	2,159,177	107.5	232,942	111,700	4.7	1,076,949
Wyoming	2,447,097	1,416,767	255.6	1,290,812	777,929	141.1	201,870	124,730	24.4	208,179
District of Columbia	640,163	311,939	11.5	431,329	212,667	6.4	293,360	144,695	4.8	1,158,907
Hawaii				947,719	471,880	20.0	130,840	65,420	.3	87,700
Puerto Rico										
TOTALS	174,836,468	91,138,605	7,409.2	155,117,663	77,497,843	4,930.5	73,228,363	34,537,297	2,282.9	76,766,151

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31, 1938

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,923,416	$ 3,874,665	136.9	$ 250,100	$ 243,478	7.8				$ 32,972
Arizona	2,569,841	3,153,137	2,501,555	193.7	38,548	38,548					29,738
Arkansas	3,352,061	3,146,869	3,117,422	350.5	181,216	180,523	9.7				54,115
California	7,747,928	7,950,258	7,534,238	263.9	138,156	138,156					75,534
Colorado	3,395,263	2,382,800	2,290,828	99.4	89,597	89,596	6.0	$ 8,200	$ 8,200		1,006,639
Connecticut	1,418,709	1,190,950	1,098,858	17.9	232,031	206,330	4.5	124,130	64,435	0.2	49,086
Delaware	900,310	871,470	843,920	66.4	10,234	10,234	.2	26,712	26,712	4.4	19,444
Florida	2,597,144	2,603,553	2,529,126	99.1	38,957	38,957					29,061
Georgia	4,988,967	1,903,869	1,860,597	109.2	2,288,924	2,020,544	105.6	442,230	442,230	29.6	665,596
Idaho	2,222,747	2,273,614	2,167,698	185.9	33,341	33,341					21,707
Illinois	8,694,009	8,139,409	7,875,966	465.0	685,010	685,010	23.2				133,033
Indiana	4,941,255	5,175,857	4,841,748	237.9	49,000	49,000					50,508
Iowa	4,991,664	5,241,284	4,885,935	528.3	105,902	104,865	.3				864
Kansas	4,994,975	4,679,497	4,626,032	370.6	251,823	249,651	21.6	38,500	38,500	.5	80,793
Kentucky	3,726,271	3,598,480	3,429,673	355.0	271,221	271,221	3.5				25,377
Louisiana	2,890,429	2,835,498	2,522,062	166.2	300,666	240,049	1.6	97,087	97,070	10.4	31,247
Maine	1,676,799	1,630,250	1,616,398	74.4	60,849	58,250	1.7				2,151
Maryland	1,750,738	773,435	766,648	27.2	467,536	467,536	10.0	168,198	134,366	4.3	382,189
Massachusetts	3,262,885	2,219,079	2,218,903	18.2	802,410	411,640	.2	1,136,726	568,363	.8	63,979
Michigan	6,301,414	6,645,939	5,950,255	288.6	284,921	284,921	3.3	223,938	63,746	.3	2,492
Minnesota	5,277,145	6,388,859	5,170,685	896.0	93,670	85,850	5.9				20,611
Mississippi	3,457,552	3,153,110	3,148,449	220.8	224,146	223,106	14.9	10,800	10,800	.6	75,196
Missouri	6,012,652	5,284,555	5,136,674	776.9	790,784	736,082	.7	34,391	32,294		107,602
Montana	3,676,416	3,595,845	3,532,169	200.7	95,385	95,385	.1	8,462	8,462		40,401
Nebraska	3,870,739	3,432,515	3,322,815	362.3	445,830	445,830	8.3	70,270	70,270	1.8	31,824
Nevada	2,243,074	2,302,820	2,193,967	110.1	84,970	38,146	1.7				10,961
New Hampshire	945,225	875,636	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,355,077	1,334,220	29.4	1,754,189	1,754,189	6.1	34,468	34,468		6,928
New Mexico	2,871,397	2,810,989	2,805,816	213.7	43,071	43,071		14,681	12,196		10,314
New York	11,046,377	10,741,839	10,276,419	170.1	316,300	276,300	1.9	62,700	62,700	.2	430,958
North Carolina	4,720,173	4,641,280	4,572,513	285.4	108,068	108,068	5.4				39,592
North Dakota	2,867,245	2,488,588	2,440,578	378.4	107,799	107,799	1.2	286,285	286,285	36.1	32,583
Ohio	7,670,815	6,993,649	6,877,800	291.9	691,682	685,682	4.7	25,850	25,850	1.9	81,483
Oklahoma	4,580,670	4,565,484	4,322,708	400.8	239,470	239,470	7.6	8,800	5,000	.1	13,492
Oregon	3,038,642	3,202,434	2,926,575	164.6	45,580	45,580		11,846	11,846	1.0	54,642
Pennsylvania	9,347,797	7,835,325	7,275,288	264.2	2,127,068	1,870,835	21.2	36,737	36,737	.4	164,937
Rhode Island	989,208	1,113,068	989,136	18.8							72
South Carolina	2,702,012	2,356,294	2,150,784	226.2	551,692	498,752	23.4	9,664	9,664	1.5	42,811
South Dakota	2,976,454	2,689,680	2,685,957	482.6	270,437	270,437	22.0				20,060
Tennessee	4,192,460	3,531,595	3,480,417	135.3	679,103	679,103	15.5	35,710	22,060	2.6	10,881
Texas	11,989,350	12,576,411	11,520,967	1,105.8	518,808	406,058	14.9	23,100	23,100	4.3	39,224
Utah	2,067,154	2,152,015	1,928,492	207.7	112,055	112,055	.1				26,607
Vermont	924,306	1,059,845	908,301	23.2	11,100	11,100					4,905
Virginia	3,652,667	3,479,593	3,290,628	941.2	131,731	128,434	13.5				233,605
Washington	3,026,161	3,316,821	2,940,420	164.3	65,824	65,824	.7				19,916
West Virginia	2,231,412	2,009,315	1,913,773	86.0	375,022	317,639	14.5				
Wisconsin	4,823,884	5,229,337	4,727,232	343.4	91,678	90,200	.3				6,452
Wyoming	2,219,155	2,171,792	2,165,171	152.4	33,287	33,287					20,697
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	946,320	862,482	17.4	9,533	8,907		62,530	54,644	.6	
TOTALS	195,000,000	183,588,755	173,245,712	12,770.4	16,719,816	15,300,340	389.5	3,002,015	2,149,998	101.6	4,303,950

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Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

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CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31, 1938

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,546,325	$ 3,530,996	47	1	12	$ 378,519	$ 378,519	2			$ 175,300	$ 109,000	4			$ 16,102
Arizona	1,256,099	1,235,065	1,216,596	15		5	18,841	18,841									20,662
Arkansas	3,574,060	2,880,768	2,873,485	51	6	27	643,751	643,046	5		4	6,541	6,541			2	50,988
California	7,486,362	7,198,713	7,019,499	46	8		414,005	414,004	1			10,000	10,000	5			42,859
Colorado	2,631,567	2,281,830	2,212,475	27	1	18	418,484	418,484	3	2	3						608
Connecticut	1,712,884	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	130,000	1			279,052	279,052	2								9,187
Florida	2,827,883	2,409,152	2,381,150	30	5	2	119,018	119,018	1			136,010	136,010			74	191,705
Georgia	4,395,949	753,574	751,468	17	6	11	1,491,539	1,491,539	23	7	6	753,110	753,110	10	2	35	1,899,832
Idaho	1,674,479	1,392,801	1,366,439	20	3	12	252,696	252,696	2		6	4,261	4,261			3	51,083
Illinois	10,307,184	8,792,358	8,668,197	65	8		1,500,134	1,493,730	9			101,000	101,000	2			44,257
Indiana	5,111,096	4,368,156	4,235,087	39	13		857,447	857,447	3		162						18,563
Iowa	5,600,679	4,583,906	4,490,835	98	9	8	1,092,875	1,089,000	8								20,844
Kansas	3,246,258	3,946,372	3,923,996	54		5	1,273,119	1,253,704	4	1		115,024	22,020	2		3	46,538
Kentucky	3,672,387	1,358,467	1,339,494	19	5		1,984,219	1,694,487	5	6		618,579	612,579	3	1		25,827
Louisiana	3,213,467	1,466,944	1,466,936	5	1		982,285	982,265	10	1		649,599	600,110	4	1		164,155
Maine	1,426,861	1,266,779	1,263,012	1	1	4	160,788	142,540	1	1							21,309
Maryland	2,061,751	665,089	660,481	5	3	18	878,674	828,674	5	2	10	279,561	257,093	1	1	2	315,503
Massachusetts	4,210,833	2,951,633	2,949,199	22	4		841,150	841,150	4			249,991	249,991	1			170,493
Michigan	6,765,197	6,861,708	6,563,144	44	8		101,477	101,477				167,000	75,730	1	1		24,846
Minnesota	5,395,441	4,753,430	4,645,372	82	13	50	676,480	670,430	4								79,639
Mississippi	3,241,475	2,326,477	2,321,323	51	5	10	448,900	448,900	6	2	3	27,700	27,700			10	443,552
Missouri	6,142,153	4,265,566	4,094,182	39	1		2,029,178	2,029,178	10		1	1,650	1,650			1	17,143
Montana	2,722,327	2,556,515	2,513,626	37	7		245,576	185,931	1	1							22,770
Nebraska	3,556,441	2,893,042	2,849,115	76	3	21	470,813	470,813	5		1	199,683	199,683	3		2	36,831
Nevada	887,260	877,030	845,455	8	3		13,308	13,308				3,630	3,630			5	24,867
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,051,471	3,038,291	20	5	1	831,570	831,570	4	4		44,730	44,730		1		69,235
New Mexico	1,725,286	1,700,885	1,693,330	19	1		25,879	25,879									6,076
New York	13,577,189	11,566,131	11,236,557	35	45		2,016,770	2,016,770	10	4		94,000	94,000				229,862
North Carolina	4,823,958	3,637,396	3,617,250	50	18		1,081,059	1,081,059	12		115						125,649
North Dakota	3,207,473	2,817,479	2,813,235	56	4		389,012	389,012		1							5,226
Ohio	8,439,897	2,496,894	2,364,902	19	6	5	5,668,434	5,200,288	30	2	6	674,350	659,350	10		3	215,357
Oklahoma	5,004,711	3,692,565	3,677,928	58	9	16	1,358,994	1,225,894	7	3	15	31,050	31,050			10	69,839
Oregon	2,334,204	2,329,684	2,240,979	16	6	2	74,215	74,215	1								19,010
Pennsylvania	11,483,613	8,099,993	7,587,065	68	18	9	3,825,861	3,458,873	18	5		150,473	150,473	5			287,202
Rhode Island	699,691	702,023	694,241	4	3												5,450
South Carolina	3,059,956	1,753,597	1,726,238	37	11	26	859,783	859,783	9	5	10	142,578	142,578	1		12	331,358
South Dakota	3,249,086	2,857,287	2,855,689	59	6	53	175,186	175,186	6			269,350	210,393	17			7,818
Tennessee	3,903,979	1,815,998	1,807,317	30	3	30	1,746,370	1,746,370	15	1	4	114,020	114,020	1		6	236,272
Texas	10,855,982	9,840,502	9,827,676	126	14	97	565,611	565,611	2	1	16	104,519	104,519			36	358,175
Utah	1,230,763	1,203,470	1,196,574	17	1	1	18,461	18,461									15,728
Vermont	729,857	752,107	706,907	10	7	20	10,900	10,900									12,050
Virginia	3,774,287	2,807,710	2,671,298	43	18	21	1,065,684	1,059,059	9	3	1	6,750	6,750			2	37,180
Washington	3,095,041	2,713,500	2,690,310	22	11	11	368,494	368,069	1	1							36,662
West Virginia	2,677,937	1,218,521	1,216,553	11	2	5	1,379,082	1,378,581	15	1	1	78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,544,529	4,508,123	37	6	7	392,238	392,035		1	10	94,010	26,563			17	95,962
Wyoming	1,360,841	1,220,886	1,203,889	13			111,212	111,212	1								45,740
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			179,710	169,698	2								
TOTALS	196,000,000	148,608,330	145,651,394	1695	313	514	40,927,221	39,441,638	264	55	374	5,299,430	4,835,495	72	7	225	6,071,473

Clemson University
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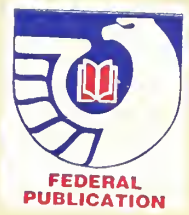

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